भारतीय मानक
Indian Standard

IS 17740 : 2022

सरंचनात्मक स्टील के एकाकी टॉवर, मस्तूल और खंभे — रीति संहिता

Isolated Towers, Masts and Poles Using Structural Steel — Code of Practice

ICS 91.080.13

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December 2022 **Price Group 15**

Structural Engineering and Structural Sections Sectional Committee, CED 07

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Structural Engineering and Structural Sections Sectional Committee had been approved by the Civil Engineering Division Council.

Towers, masts and poles are tall structures usually of steel construction. They are used to support wide variety of radio, TV broadcasting, antenna and other communication equipment or flood lighting. These towers are key infrastructural components and must perform many functions satisfactorily. Amongst these functions are, the utility of the towers for broadcasting, mobile network and flood lighting, while meeting the structural safety and serviceability limit state requirements. The designs of the towers depend upon the minimum requirements prescribed for each one of the above functions.

This standard has been brought out for providing guidance for safe design, fabrication and erection of towers, masts and monopoles using structural steel. This standard covers all type of freestanding towers including communication towers (broadcasting and tele-communication), antenna supporting structures and flood lighting structures. These are self-supporting lattice towers, guyed masts or steel monopoles, covering their materials, loads and design. The provision for fabrication, erection, inspection and testing of these towers are covered in IS 800 : 2007 ‘General construction in steel — Code of practice’ and IS 802 (Part 3) : 1978 ‘Code of practice for use of structural steel in overhead transmission line towers: Part 3 Testing’.

The steel towers or masts are comparatively light structures. Hence, the wind load acting on them is usually the main criterion for their design. Since the simultaneous occurrence of earthquake and maximum wind pressures are unlikely to take place, the provisions of earthquake forces have not been specified in this standard. The provisions of earthquake forces have not been specified in this standard. To account for the effects of earthquake ground shaking, reference may be made to IS 1893 (Part 4) : 2005 ‘Criteria for earthquake resistant design of structures: Part 4 Industrial structures’.

The standard envisages for the present, the fabrication of these towers by means of bolted and shop welded connections only. The design provisions, when not covered in this standard may generally be adopted in accordance with IS 800.

Ice loadings on towers and equipment located in the cold mountainous regions of the country subject to snow fall, may be taken into account on the basis of available meteorological data, with ice as well as with wind and without wind.

Compliance with this standard does not relieve the user from the responsibility of observing state/local building regulations, fire and safety regulations, and civil aviation requirements, pertaining to such structures.

In the formulation of this standard, assistance has also been derived from the following publications:

ASCE 52 : 1988	Guide for design of steel transmission towers, American Society of Civil Engineers
ANSl/TIA-222-G : 2006	Structural standard for antenna supporting structures and antennas
CANCSA-S37-M86 : 1986	Antennas, towers, and antenna–supporting structures, Canadian Standards Association
ASCE/SEI 48 : 2011	Design of steel transmission line pole structures, American Society of Civil Engineers
AS 3995 : 1994	Design of steel lattice towers and masts, Standard Australia
DIN 4131 : 1991	Steel radio towers and masts, German Institute for Standardisation
ASCE 10 : 2015	Design of latticed steel transmission structures, American Society of Civil Engineers

The composition of the Committee responsible for the formulation of this standard is given in Annex M.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall he rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

ISOLATED TOWERS, MASTS AND POLES USING STRUCTURAL STEEL — CODE OF PRACTICE

1 SCOPE

1.1 This standard covers the structural design for static and dynamic wind forces, fabrication, and erection of all types of isolated steel towers including communication towers, antenna supporting structures and flood lighting structures, consisting of:

a) Self-supporting lattice towers,
b) Guyed masts, and
c) Steel monopoles.

1.2 The requirements of number, height, size, structural layout, location and spacing of towers arising out of mobile network planning considerations are not covered in this standard.

1.3 This standard also does not deal with the packing material, antenna types, spacing of towers from buildings or human habitat and the method of testing the performance requirements of communication structures.

1.4 This standard does not apply to structures with unconventional shapes, unusual locations, and materials other than steel as well as abnormal environmental conditions that have not been covered in this standard. Special investigations are necessary in such cases to establish loads and design procedures.

1.5 In the cases of structures with unsymmetrical geometry, the checking may have to be carried out for special wind load effects that are not covered in this standard.

2 REFERENCES

The standards given in Annex A contain provisions, which through references in this text, constitute provisions of this standard. All standards are subjected to revision, and parties to agreement based on this standard are encouraged to apply the most recent edition of the standard indicated.

3 TERMINOLOGY

For the purpose of this standard the following definitions shall apply.

3.1 Actual Length — The length between centre-to-centre of intersection points, with supporting members or the cantilever length in the case of a freestanding member.

3.2 Angle of Attack — An angle between the direction of wind and a reference axis of the structure in plan.

3.3 Antenna Supporting Structure — A structure, including guy assemblies, guy anchorages and substructures, that supports antennas or antenna arrays.

3.4 Antenna — A conductor for radiating or receiving radio/electromagnetic waves exclusive of the connecting wires between its main portion and the appurtenances associated with it, and shall include any reflector, director and screen designed to produce a specific radiation pattern.

3.5 Appurtenances — Items attached to the structure, such as antennas, antenna mounts, antenna cables, conduits, lighting equipment, climbing devices, platforms, signs, anti-climbing devices etc.

3.6 Bearing Type Connection — A connection made using bolts in snug-tight condition, or rivets where the load is transferred by bearing of bolts or rivets against plate inside the bolt hole.

3.7 Braced Member — A member in which the relative transverse displacement is effectively prevented by bracing.

3.8 Breadth — The horizontal dimension of the tower like structures, measured normal to the direction of wind.

3.9 Dead Load (DL) — The weight of the structure, and appurtenances, excluding guy assemblies and for foundation design, the weight of the soil and sub structure.

3.10 Depth — The horizontal dimension of the tower like structures measured in the direction of the wind.

3.11 Design Life/Mean Probable Design Life — Time period for which a structure or a structural element is required to perform its function without damage.

3.12 Design Load/Factored Load — A load value obtained by multiplying the characteristic load with a load factor.

3.13 Design Strength — The nominal strength divided by resistance factor.

3.14 Design Wind Load — Equivalent static force to be used in the determination of wind loads.

3.15 Developed Height — The height of upward penetration of the velocity profile in a new terrain. At large fetch lengths, such penetration reaches the gradient height, above which the wind speed may be taken to be constant. In case of lesser fetch lengths, a velocity profile of a smaller height but similar to that of the fully developed profile of that terrain category has to be taken, with the additional provision that the velocity at the top of this shorter profile equal to that of the un-penetrated earlier velocity profile at that height.

3.16 Discrete Appurtenances — An appurtenance modelled as a concentrated load.

3.17 Displacement — It is the deviation or deformation from the standard position of a member or structure due to service loads.

3.18 Effective Frontal Area — The projected area of the structure normal to the direction of wind.

3.19 Effective Lateral Restraint — Restraint that produces sufficient resistance to prevent deformation in the lateral direction.

3.20 Effective Length — The actual length of a member between points of effective restraint or effective restraint and free end, multiplied by a factor to take account of the end conditions in buckling strength calculations.

3.21 Effective Length Factor — A factor, K, to modify the unbraced length, L, to take into account the structural configuration and end conditions.

3.22 Effective Slenderness Ratio — The slenderness ratio modified to take into account the structural configuration and end conditions for the purpose of calculating the design compression strength.

3.23 Element of Surface Area — The area of surface over which the pressure coefficient is taken to be constant.

3.24 Escarpment — A cliff or steep slope generally seperating two levels or gently sloping areas.

3.25 Factor of Safety — The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

3.26 Fetch Length — The distance of the tower measured along the wind from a boundary at which a change in the type of terrain occurs. When the changes in terrain types are encountered (such as, the boundary of a town or city, forest), the wind profile changes in character but such changes are gradual and start at ground level, spreading or penetrating upwards with increasing fetch length.

3.27 Force Coefficient — A non-dimensional coefficient such that the total wind force on a body is the product of the force coefficient, the dynamic pressure of the incident design wind speed and the reference area over which the force is required. When the force is in the direction of the incident wind, the non-dimensional coefficient is called as drag coefficient. When the force is perpendicular in plan to the direction of incident wind, the non-dimensional coefficient is called as 'lift coefficient'.

3.28 Friction Type Connection — Connection effected by using pre-tensioned high strength bolts where shear force transfer is due to mobilization of the friction between the connected plates due to clamping force developed at the interface of connected plates by the bolt pre-tension.

3.29 Glaze Ice — The ice accretion, that is assumed to have a unit weight of 8.83 kN/m^3.

3.30 Gradient Height — It is the height above the mean ground level at which the gradient wind blows as a result of balance among pressure gradient force, coriolis force and centrifugal force. For the purpose of this standard, the gradient height is taken as the height above the mean ground level, above which the variation of wind speed with height need not be considered.

3.31 Ground Roughness — The nature of the earth's surface as influenced by small scale obstructions such as trees and buildings (as distinct from topography).

3.32 Gust — A positive or negative departure of wind speed from its mean value, lasting for not more than, say 2 min over a specified interval of time.

3.33 Guy Assembly Dead Load (D_g) — The weight of all guy assemblies including guys and end fittings.

3.34 Guy Assembly Link Plate — A plate connecting two guy assemblies or a guy assembly to an anchorage or attachment point.

3.35 Guyed Mast — A latticed or pole structure with supporting guys anchored to the supporting medium.

3.36 Height of the Structure 'h' — The total height of a structure above mean ground, including latticed or tubular poles mounted on the structure, but excluding lightning rods and similar appurtenance.

3.37 Hill — A land surface characterized by strong relief in all horizontal directions.

3.38 Limit State — A limiting condition beyond which the structure ceases to fulfil its intended function.

3.39 Line of Action — A line that is parallel to the longitudinal axis of a member that passes through the centroid of the bolt group connecting the member to another member.

3.40 Linear Appurtenances — An appurtenance modelled as distributed load.

3.41 ln-plane Buckling — The buckling in which direction of buckling is considered in the plane defined by the face of a latticed structure.

3.42 Load Factor — A factor that accounts for deviations of the actual load from the nominal load for uncertainties in the analysis that transforms the load into load effects, and for the probability that more than one extreme load will occur simultaneously.

3.43 Main Member — A structural member, which is primarily responsible for carrying and distributing the applied load or action.

3.44 Mast Span — The distance between the base and the first guy level, the distance between two successive guy levels, or the distance above the top guy level to the top of the structure (cantilever span).

3.45 Mean Ground Level — The average horizontal plane of the area enclosed by the boundaries of the structure.

3.46 Nominal Strength — The capacity of a structure or member to resist the effects of loads.

3.47 Out-of-Plane Bucking — Direction of buckling considered normal to the face of a latticed structure.

3.48 Panel Points — The centreline location of bracing member connections to a leg.

3.49 Peak Gust or Peak Gust Speed — The wind speed associated with the maximum short duration wind gust.

3.50 Pressure Coefficient — The ratio of the design wind pressure acting at a point on the surface and the static pressure of the incident wind, where the static and design wind pressures are determined at the height of the point considered after taking into account the geographical location, terrain conditions and shielding effect. The pressure coefficient is also equal to $[1 - (V_p/V_z)^2]$, where V_p is the actual wind speed at any point of the structure at a height corresponding to that of V_z.

NOTE — Positive sign of the pressure coefficient indicates pressure acting towards the surface and negative sign indicates pressure acting away from the surface.

3.51 Radius of Gyration — The square root of the moment of inertia about the axis of buckling under consideration divided by the area of a member.

3.52 Required Strength — The sum of the load effects due to applied factored loads and load combinations.

3.53 Return Period — The mean interval between recurrences of a climatic event of defined magnitude. The inverse of return period gives the probability of exceeding the event in one year.

3.54 Ridge — An elongated crest characterized by strong relief in two direction.

3.55 Safety — The ability of a system not to cause human injuries or loss of life. It relates, in this standard, mainly to protection of workers during construction and maintenance operations.

3.56 Secondary Members — Members used primarily to reduce the unbraced length of a supported member.

3.57 Service Loads — The load combination used to calculate serviceability limit state performance.

3.58 Shielding Effect (or Shielding) — The condition where wind has to pass along some structures(s) or structural element(s) located on the upstream wind side, before meeting the structure or structural element under consideration. A factor called shielding factor, is used to account for such effects in estimating the wind force on the shielded structure(s) or shielded element(s).

3.59 Slenderness Ratio — The ratio of the unbraced length, L, to the corresponding radius of gyration, r.

3.60 Slip-Critical Connection — A bolted joint governed by shear using oversized or slotted holes parallel to the line of force.

3.61 Solidity Ratio — The effective project area of all the individual elements of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

NOTE — Solidity ratio is to be calculated for individual frames.

3.62 Strength Design — Method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the design strengths of the members.

3.63 Suction — The phenomenon when pressure is less than the atmosphere (static) pressure and is taken to act away from the surface.

3.64 Sway — The angular rotation under service loads of an antenna beam path in the local vertical plane of the antenna from unfactored no-wind load position.

3.65 Symmetrical Appurtenance — An appurtenance for which the effective projected area is considered constant for all wind directions.

3.66 Terrain Category — The category of land type indicating the roughness characteristics of the ground surface irregularities of an area, which arise from natural or constructed features. The categories are numbered in increasing order of roughness.

3.67 Topography — The nature of the earth's surface as influenced by the hill and valley configurations.

3.68 Twist — The angular rotation under service loads of an antenna beam path in the local horizontal plane of the antenna under unfactored no-wind load position.

3.69 Unbraced Length, *L* — The length between panel points or nodes providing restraint, which may vary for different planes of buckling depending on the bracing pattern. For leg members, *L* shall not be less than the centreline distance between panel points. For bracing members, *L* shall not be less than the length between the centres of connecting bolt or weld patterns.

3.70 Velocity Profile — The variation of the horizontal component of the atmospheric wind speed at different heights above the mean ground level.

3.71 Weak Axis Buckling — The direction of buckling considered about the weak principal axis of a member.

4 SYMBOLS AND NOTATIONS

Symbols and notations used in this standard have been explained in this section or in respective sections, as applicable. Detailed notation information shall be sought from the reference standards, as mentioned:

A = Effective net area

a = End distance of a guy assembly link plate

A_b = Nominal unthreaded body area of a bolt or anchor rod

A_e = Equivalent slenderness ratio

A_{en} = Effective net area

A_g = Gross cross-sectional area

A_{go} = Gross area of the outstanding leg

A_{gt} = Gross area subject to tension

A_{gv} = Gross area subjected to shear

a_i = Spacing of intermediate connectors for a built-up member

A_n = Net area of a member, bolt or anchor rod, Net area of the total cross-section

A_{nc} = Net area of the connected leg

A_{nt} = Net area subjected to tension

A_{nv} = Net area subjected to shear

A_p = Cross section area of removed for an opening in a tubular section

A_r = Cross section area of reinforcing for an opening in a tubular section

A_{sf} = Effective shear area of a guy assembly link plate

A_{tg}, A_{tn} = Minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force respectively

A_{vg}, A_{vn} = Minimum gross and net area in shear along line parallel to external force respectively

B = Width of an angle leg, Width of the plate

B_1 = Moment amplification factor

b_{eff} = Effective edge distance of a guy assembly link plate

C_f = Force coefficient or drag coefficient

C_t = Torsional constant

D = Diameter of cylinder; outer diameter of a round member or outside flat width of a polygon member

d = Nominal diameter of the bolt, pin or anchor rod

D_h = Diameter of the bolt hole

d_n = Root diameter of a threaded anchor rod

E = Modulus of Elasticity of steel = 205 000 MPa

$E\mathrm{I}$ = Flexural stiffness

F = Force normal to the surface

F'_y = Effective yield stress

f_{cc} = Euler buckling stress = $\pi^2E/(KL/r)^2$

f_{cd} = Design compressive stress

F_{cr} = Critical compression stress

F_e = Elastic buckling stress

F_{ey} = Elastic buckling stress about the principal axis of a double angle member

F_{ez} = Torsional elastic buckling stress

F_{nt} = Critical shear stress for tension

F_{nv} = Critical shear for direct tension

F_s = Axial design compressive force in a supported member

f_u = Characteristic ultimate tensile stress

F_u = Specified minimum tensile strength

i_{ub} = Specified minimum tensile strength of a bolt or anchor rod

f_y = Characteristic yield stress

F_y = Specified minimum yield strength

F_{yr} = Specified minimum yield strength of reinforcement for an opening in a tubular section

g = Gauge length between the bolt holes

G = Shear modulus of elasticity, 77000 MPa

g_c = Distance from the heel of an angle to the end connection centroid

H = Flexural constant

h = Height of development of a velocity profile at a distance x downwind from a change in terrain category

H_1 = Unbraced length for investigating buckling for horizontal bracing number

I_{ar} = Anchor rod projection

I_x, I_y = Moment of inertias about the principal axes

J = St. Venant torsional constant

K = Effective length factor

k_1 = Probability factor (risk coefficient)

k_2 = Terrain, height and structure size factor

k_3 = Topography factor

k_4 = Cyclonic factor

K_i = Effective length factor for the individual members of a built-up member

KL = Effective length

KL/r = Effective slenderness ratio

$(KL/r)_m$ = Modified effective slenderness ratio of a built-up member acting as a unit

$(KL/r)_o$ = Effective slenderness ratio of a built-up member acting as a unit

L = Laterally unbraced length of a member

L/r = Slenderness ratio of a member

L_1, L_2, L_3 = Unbraced lengths for investigating buckling for diagonal bracing members

L_b = Connection length for a multiple bolt connection

L_c = Clear distance for design bearing strength

L_e = Length of member or greater horizontal dimension of a building

L_p = Length of a tubular structure considered for shear strength

m = Torsional constant shape coefficient for a member resisting torsion

M_n = Nominal flexural strength

M_u = Flexural moment due to factored loads

n = Number of threads per inch

N = Staggered bracing panel spacing ratio

P = Factored applied axial force

p = Pitch of the threads

P' = Staggered - pitch length between the bolt holes

P_d = Design axial compressive strength

P_e = Elastic critical buckling load

P_n = Nominal axial strength

P_r = Resistance required to provide lateral support at a panel point within a face of a latticed structure

P_s = Minimum bracing resistance normal to a supported member required to provide lateral support

P_u = Axial force due to factored loads

P_{uc} = Anchor rod axial compression force due to factored loads

P_{ut} = Anchor rod axial tension force due to factored loads

P_z = Wind pressure at height, z in N/m^2

r = Radius of gyration

R_b = Connection length reduction factor

r_i = Minimum radius of gyration of an individual component a built-up member

r_{ib} = Radius of gyration of an individual component about its centroid axis in the

r_{min} = Minimum radius of gyration for a member

R_n = Nominal bearing strength at a bolt or attachment hole

R_{nb} = Nominal buckling strength of an anchor rod acting as column

R_{nc} = Nominal compression yield strength of an anchor rod

R_{nk} = Nominal strength of a guy assembly link plate

R_{np} = Nominal strength of a connecting element

R_{nr} = Nominal torsional strength (twisting) of a U-bolt connection

R_{ns} = Nominal sliding strength of a U-bolt connection

R_{nt} = Nominal tensile strength of a bolt or anchor rod

R_{nv} = Nominal shear rupture strength of a bolt or anchor rod

R_{nvc} = Nominal shear yield strength of an anchor rod

$\bar{r}_o$ = Polar radius of gyration about the shear center

r_{out} = Radius of gyration associated with out of-plane buckling with respect to the face of a latticed structure

r_x = Radius of gyration about the x-axis of buckling

r_y = Radius of gyration about the y-axis of buckling

r_z = Radius of gyration about the z-axis of buckling

s = Longitudinal centre-to-centre spacing (pitch) of any two consecutive holes

S = Elastic section modulus

t = Thickness of a member or connected part

T_d = Design strength under axial tension

T_{db} = Design strength of bolt under axial tension: Block strength at end connection

T_{dg} = Yielding strength of grass section under axial tension

T_{dn} = Rupture strength of net section under axial tension

T_n = Nominal torsional strength

T_p = Installed pretention in each leg of a U-bolt connection

T_u = Torsional moment due to factored loads

T_{ub} = Bolt tensile force due to factored loads

T_{ur} = Torsional moment (twisting) applied to a member from a u-boil connection due to factored loads

T_{ut} = Tension force applied to a U-bolt assembly due to factored loads

U = Reduction factor for effective net area calculation

U_{bs} = Reduction coefficient for block shear rupture

V = Design wind velocity in at height

V_d = Regional basic wind speed

V_n = Nominal shear strengths

V_u = Transverse shear force due to factored loads

V_{ub} = Bolt shear force due to factored loads

V_{us} = Sliding force applied to a member from a U-bolt connection due to factored loads

W = Outstand leg width

w = Width of the flat member

W_n = Net width for area calculation

X = Stress reduction factor ratio

X_f = Location of reinforcement for an opening in a tubular section from the section center line

γ_{m1} = Partial safety factor against ultimate stress = 1.25

γ_{mo} = Partial safety factor against yielding stress and buckling = 1.10

y_o = Coordinates of shear center with respect to the centroid of a member

Z = Plastic section modulus

ϵ = Yield stress ratio $(250/f_y)^{0.5}$

λ = Non - dimensional effective slenderness ratio

Φ = Resistance factor for connections

Φ_a = Resistance factor for normal stress

Φ_c = Resistance factor for compression

Φ_f = Resistance factor for flexure

Φ_k = Resistance factor for guy assembly link plates

Φ_p = Resistance factor for connecting elements

Φ_r = Resistance factor for torsion

Φ_t = Resistance factor for tension

Φ_u = Resistance factor for U-bolts

Φ_v = Resistance factor for shear

θ = Angle between the two longitudinal axis of two structural members of a latticed structure

5 MATERIALS

5.1 Structural Steel

The tower members including plates shall be of structural steel conforming to any of the grades of IS 2062, as appropriate.

5.2 Bolts

Bolts for tower connections shall conform to IS 12427 or property class 4.6 or 5.6, 6.8 conforming to IS 6639. High strength bolts, if used shall conform to property class 8.8 and 10.9 of IS 3757. Foundation bolts shall conform to IS 5624.

5.3 Nuts

Nuts shall conform to IS 1363 (Part 3). The mechanical properties shall conform to property class 4 or 5 as the case may be as specified in IS 1367 (Part 6) except that the proof stress for nuts of property class 5 shall be as given in IS 12427. Nuts to be used with high strength bolts shall conform to IS 6623.

5.4 Washers

Washers shall conform to IS 2016. Heavy washers shall conform to IS 6610. Spring washers shall conform to type B of IS 3063. Washers to be used with high strength bolts and nuts shall conform to IS 6649.

5.5 Galvanization

Structural members of the towers, plain and heavy washers shall be galvanized in accordance with the provision of IS 4759. Threaded fasteners shall be hot dip galvanized to conform to the requirements of IS 1367 (Part 13). Spring washers shall be hot dip galvanized as per service Grade 4 of IS 4759 or electro-galvanized as per service Grade 3 of IS 1573 as specified by the purchaser.

5.6 Guys and Shackles

Material used in guys, other components of guy system and their behaviour are briefly discussed in Annex B.

5.7 Other Materials

Other materials used in the construction of the tower shall conform to appropriate Indian Standards as available.

6 CLASSIFICATION OF STRUCTURE

Structures shall be classified under different risk categories as given in Table 1, according to the location and purpose of use, for determining nominal wind and ice loads and the load factors to be used.

Table 1 Risk Categorization of Structures

(*Clause* 6)

Sl No.	Use of Description of Structure	Risk Category
(1)	(2)	(3)
i)	Towers and masts erected in urban locations, or where their failure is likely to cause injury or loss of life; towers and masts used for vital telecommunication facilities; other major structures where the consequences of failure would be likely to be very high. Towers and masts in this category are those that in the event of failure would threaten the life line functionality or integrity of facilities with the potential to cause mass disruption (loss of power, transportation, water, etc.) of day to day civilian life in the event of failure. Structures in this category are used for communication across non-redundant and hardened networks such as; civil or national defence; rescue or disaster operations; military and navigation facilities.	III
ii)	All towers and masts that cannot be defined as class I or III.	II
iii)	Towers and masts built on unmanned sites in open countryside; towers and masts, the failure of which would not be likely to cause injury to people and/or damage to surrounding facilities in the event of failure. Towers and masts in this category are used for services that are optional and/or where an extensive delay in returning the services would be acceptable such as: redundant wireless antennas; lower-power radio access nodes (small cell); single-appurtenance supporting structures that allow for rapid repair or replacement, residential wireless and conventional 2-way radio communications, television, radio and scanner reception; wireless cable; amateur and CB radio communications.	I

7 LOADS

7.1 General

Loads in general shall be according to the relevant provisions of IS 875 series of standards, unless otherwise specified herein under.

7.2 Types of Loads

For the purpose of designing of a tower or a structure, the following loads are taken into account:

a) Dead load of the system;
b) Loads resulting from attachment of supplementary;
c) Live loads;
d) Snow load, if applicable;
e) Wind loads;
f) Seismic load; and
g) Any other loads as specified (wind on ice).

7.2.1 *Dead Load of the System*

The dead loads are the weight of the structure, and other superimposed dead loads, such as antenna supporting fixtures, platforms, ladders, attachments/appurtenances; the weight of the antennas, antenna mounts, power/signal cables, conduits, lighting equipment, climbing devices, sign boards, anti-climbing devices etc.

7.2.2 *Live Loads*

On ladders, platforms, etc, a uniformly distributed imposed load of 1500 N/m^2 shall be assumed. This also applies for the surfaces of bases with aerials. The individual parts of the ladders, platforms, etc shall also be checked for a single concentrated load of 1330 N at the most unfavorable locations at 3 m interval. On hand railing, a horizontal load of 500 N/m and a concentrated load of 1.5 kN acting inward or outward at the most unfavourable location shall also be assumed.

7.2.3 *Snow Loads*

Snow load shall be taken as given in IS 875 (Part 4). If the data for snow loads in the zone, where the tower is to be erected, are not available and if the snow load is to be taken into account, a uniform 30 mm of icing on structural members, cables and aerials may be assumed or as specified by the authorities concerned.

7.2.4 *Design Ice Thickness*

Atmospheric ice loading on masts and towers can grow to considerable thickness, for exposed sites. Combined with wind the increased wind drag due to iced members might in some instances govern the design, even though the associated wind speeds with ice are less than the maximum wind speeds during no ice conditions. When combining ice and wind load, the magnitude of ice deposit on structures as well as the density, the placing and the shape of the ice on mast and towers heavily depends on the local meteorological conditions and topography and the shape of the structure itself.

7.2.4.1 The design ice thickness, t_{iz} shall be escalated with height when calculating ice weight and wind on ice loads in accordance with the following equations:

$$t_{iz} = 2.0\, t_i\, (k_1)^2\, k_{iz}\, (k_3)^{0.7}$$

$$k_{iz} = \left[\frac{z}{10}\right]^{0.1} \cdot \leq 1.4, \textit{ Z in metres}$$

where

t_{iz} = Factored thickness of radial glaze ice at height, Z;

t_i = Design ice thickness;

k_{iz} = Height escalation factor for ice thickness; and

Z = Height above ground level at base of structure.

The coefficient 2.0 is the limit state conversion factor

7.2.4.2 For purpose of calculating the additional projected area of ice, ice thickness shall be considered to accumulate with a uniform thickness around the exposed surfaces of the structure, guys and appurtenances (*refer* to Fig.1). The additional projected area of ice may be considered round and round member force coefficients be used, when calculating wind on ice loads even though the bare projected areas are flat.

7.2.4.3 For purposes of calculating the weight of ice, the density shall be taken as 8.83 kN/m^3 and the cross-sectional area of ice shall be determined by the following equation:

$$A_{iz} = \pi\, t_{iz}\, (D_c + t_{iz})$$

where

A_{iz} = Cross sectional area of ice at height z; and

D_c = Largest out-to-out dimension of a member (*refer* Fig. 2).

7.3 Wind Loads

7.3.1 IS 875 (Part 3) shall be referred to evaluate the basic wind pressure and the wind loads on the structure. To determine the design wind velocity, the following design life/Mean probable design life shall be used as per the risk category of the tower/mast.

Category	Design life/Mean Probable design life
I	25 years
II	50 years
III	100 years

NOTE — As per IS 875 (Part 3), only structures and ground based freestanding towers of heights of less than or equal to 50 shall be designed for Category I and II.

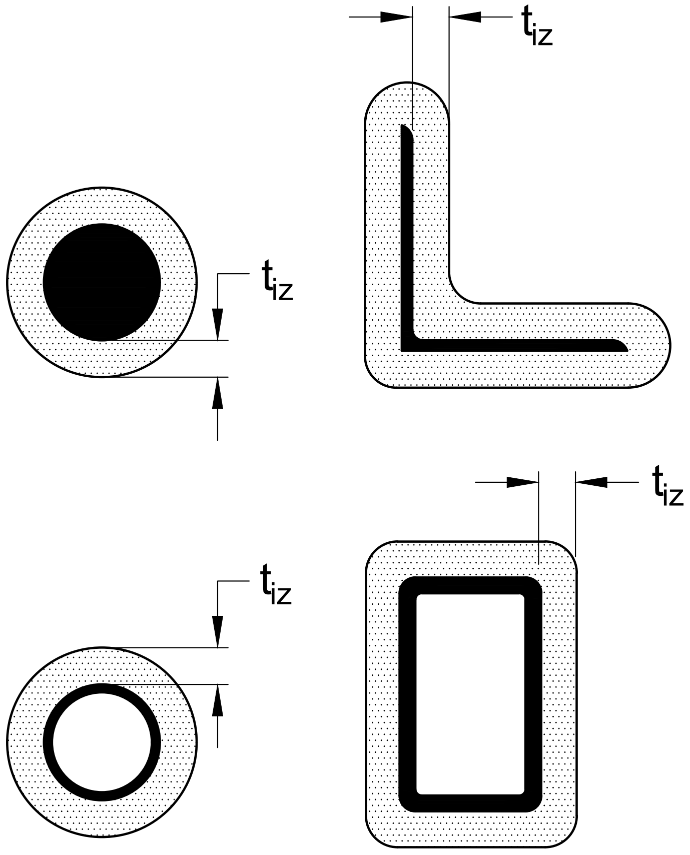
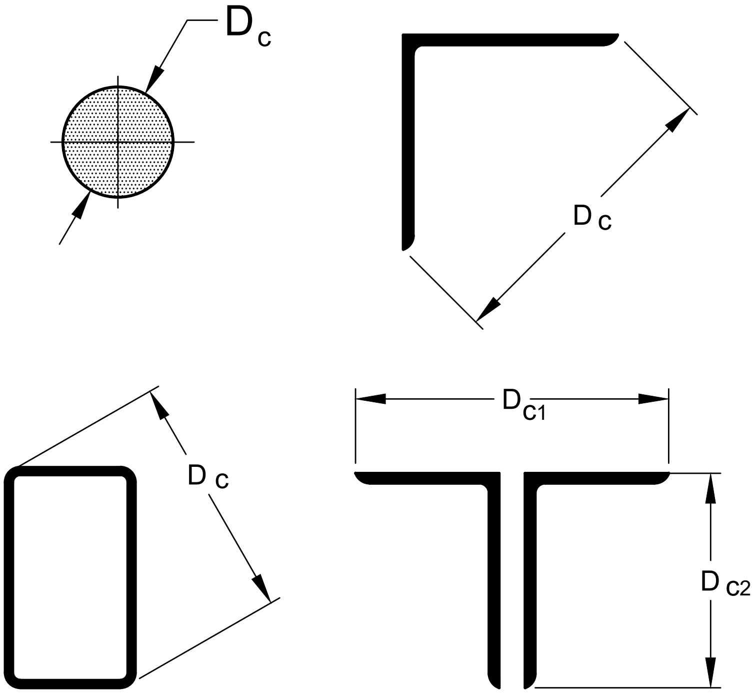


FIG. 1 PROJECTED AREA OF ICE



NOTE — D_C is the Larger of D_{C1} and D_{C2}

FIG. 2 OUT-TO-OUT DIMENSION FOR CALCULATING ICE WEIGHT

7.3.2 *Design Wind Load*

The design wind load shall include the sum of the horizontal design wind forces on the structure in the direction of the wind and the design wind forces on guy and appurtenances. All appurtenances, including antennas, mounts and lines, shall be assumed to remain intact and attached to the structure.

Design forces shall be based on the wind directions resulting in the maximum responses. For lattice structures, each of the wind directions indicated in Fig. 3 shall be considered for each face.

The design wind load, F_w, shall be determined in accordance with the following:

$$F_w = F_{ST} + F_A + F_G$$

where

F_{ST} = Design wind force on the structure [*refer* IS 875 (Part 3)] and section **7.4**;

F_A = Design wind force on appurtenances as per section **7.5**; and

F_G = Design wind force on guys as per section **7.10**.

7.4 Design Wind Force on Hybrid Tower Structure

In equilateral triangular towers with circular members for legs and flat sided members for other members, the force coefficient shall be calculated separately for both circular members and flat sided members individually by using the (same) solidity ratio of that particular panel (for example, solidity ratio for a given panel shall be calculated based on the ratio of total covered area (sum of the exposed areas of both circular and flat sided members) divided by the panel area. The total wind load ***F*** on the hybrid tower surfaces shall be calculated as given below and shall be assumed as acting horizontally:

$$F = (C_{f\,Flat} \times A_{Flat} + C_{f\,Cir} \times A_{Cir}) \times P_d$$

Where,

$C_{f\,Flat}$ = Force coefficient for flat sided members;

$C_{f\,Cir}$ = Force coefficient for circular members;

A_{Flat} = Exposed area of flat sided members;

A_{Cir} = Exposed area of circular members; and

P_d = Design wind pressure.

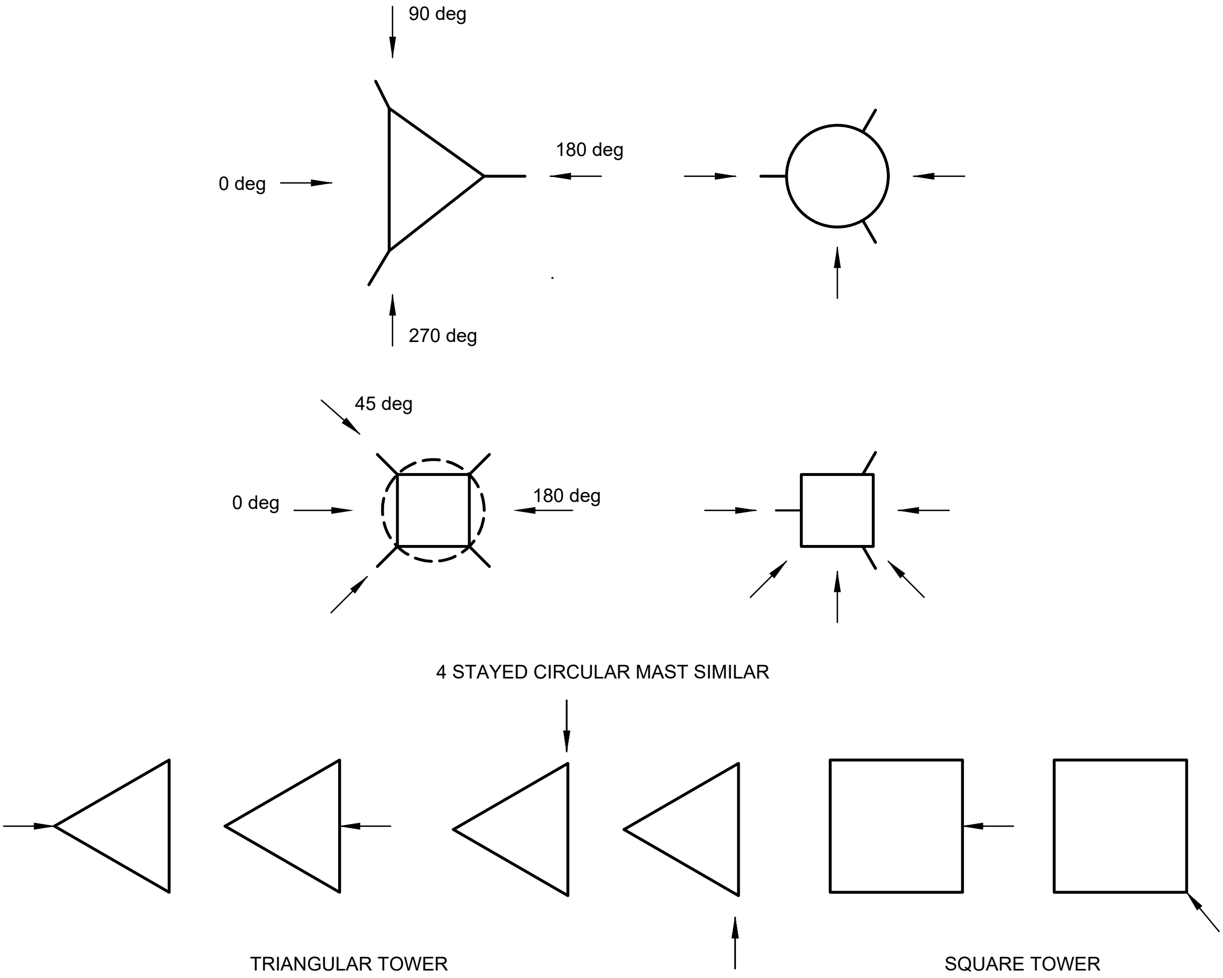
7.5 Design Wind Force on Appurtenances

The design wind force on appurtenances (either discrete or linear but excluding microwave antennas), F_A, shall be determined from the equation:

$$F_A = C_{fa} \times P_d \times A_a$$

where

P_d = design wind pressure at the centerline height of the appurtenance;



NOTES

1 Wind directions given in Fig. 3 are applicable only in case of antenna/equipment wind exposed area is symmetrically on all faces of the tower. All critical wind directions around tower need to be investigated in case of unsymmetrical exposed area on different faces of the tower.

2 The design wind forces $F_{ST} + F_A$, need not exceed the wind force calculated for a structure using a solidity ratio of 1.0 (solid faced) plus the wind load on externally mounted appurtenances that are outside the normal project area of the structure in the direction of the wind.

FIG. 3 TYPICAL WIND DIRECTIONS TO BE CONSIDERED

A_a = effective projected area of the appurtenance including ice for loading combinations that include ice. Appurtenance shall be considered as consisting of flat and round components; and

C_{fa} = drag force coefficient for appurtenances.

The design wind force, F_A, shall be applied at the centroid of the effective projected area of the appurtenance in the direction of the wind. For a linear appurtenance, the design wind pressure corresponding to the segment or panel shall be considered.

In the absence of more accurate data, the design wind force on microwave antenna shall be determined using Annex L.

NOTE — The wind load on ladders, power conductors or conduits, antennae, transmission lines, service struts etc. or those members occurring in only one face of the structure, shall be obtained by considering the unit basic wind pressure with appropriate force coefficient as acting, normal to one time, the exposed projected area, including the specified ice covering, if any, on such attachment.

7.6 Antenna Mounting Pipes

The projected area of a mounting pipe above and below an antenna shall be included in the term (A_a). The effective projected area associated with the windward face normal to the azimuth of a mounting frame A_a shall be determined.

7.7 Individual Members

The force coefficient given in IS 875 (Part 3) refers to member of infinite length. For the members of finite length, the coefficient should be multiplied by a factor *K*, given in IS 875 (Part 3), that depends on the ratio *l*/*b* where *l* is the length of the member and *b* is the width across the direction of wind.

7.8 Effective Projected Area of Pole Structures

The effective projected area (EPA) of pole section shall be determined from the equation:

$$(EPA)_s = C_f A_p$$

where

C_f = Force coefficient for cantilevered pole structure [IS 875 (Part 3)]; and

A_p = Actual projected area based on the pole outside diameter (for rounds), the outside point-to-point diameter (for polygon), or overall width, including ice thickness for load combinations that include ice.

7.9 Shielding

Shielding, except as noted herein, may be considered for intersecting or parallel elements. The unshielded element shall be considered as flat unless both elements are round. Full shielding may be considered when the clear distance, D_w, between the elements in the direction of the wind under consideration for determining effective projected areas (A_e) is less than or equal to 2.0 times the smallest projected dimension of the element in the direction normal to the direction of the wind under consideration. No shielding shall be considered when the above ratio is greater than 4.0. Linear interpolation shall be allowed for the values of the above ratio between 2.0 and 4.0 (Fig. 4).

7.10 Design Wind Force on Guys

The design wind force on guys, F_G, shall be determined using the following equation:

$$F_G = C_d \, d \, L_G P_d \, \text{Sin}^2\theta_g$$

where

F_G = Force applied normal to the chord of the guy in the place containing the guy chord and the wind, (*refer* Fig. 5);

C_d = 1.2, drag factor for the guys;

d = Guy diameter including ice for loading combinations that include ice;

L_G = Length of guy;

P_d = Wind pressure at height, z; and

θ_g = True angle of wind incidence with respect to the plane of the guy along with its chord.

NOTE — A higher drag factor, C_d, or an increase effective guy diameter may be required when attachments to guys, such as spoilers, insulators, makers etc. are present.

The design wind force and ice thickness may be assumed to be uniform based on the velocity pressure and ice thickness at the mid-height of each guy or guy segment. The length of each guy or guy segment may be assumed to equal the chord length. The design wind force shall be considered as a distributed force normal to the guy chord (*see* Fig. 5).

For ground supported structures, mid-height shall be referenced to the ground elevation at the base of the structure. For structures supported on building or other supporting structures, the mid-height of a guy shall be measured from the mid-height elevation of the guy to the ground level of the building or other supporting structure. The height for a guy segment shall not be less than zero.

8 DYNAMIC EFFECTS

8.1 Dynamic Pressure of Wind, *P*

On the basis of measured maximum wind velocities for different parts of the country including wind of short durations as in squall, the country has been divided into different wind zones (IS 875 Part 3). The dynamic wind pressure, as given in clauses **9** and **10** of IS 875 (Part 3) shall be considered.

8.2 Flexible slender structures and structural elements shall be investigated to ascertain the importance of wind induced oscillations or excitations in along wind and across wind directions.

In general, the dynamic effects of wind induced oscillations shall be considered when the tower/mast natural frequency in the first mode, f_1, is less than 1.0 Hz (or the natural time period, *T*, is greater than 1 s).

For calculation of the along wind loads and response (bending moments, shear forces, or tip deflections) the Gust Factor (*GF*) method is to be used as specified in IS 875 (Part 3).

8.3 The fundamental time period (*T*) may either be established by experimental observations on similar structures or calculated by any rational method of analysis. In the absence of such data, *T* may be determined as:

$$T = \frac{1}{f_1} \text{ in seconds}$$

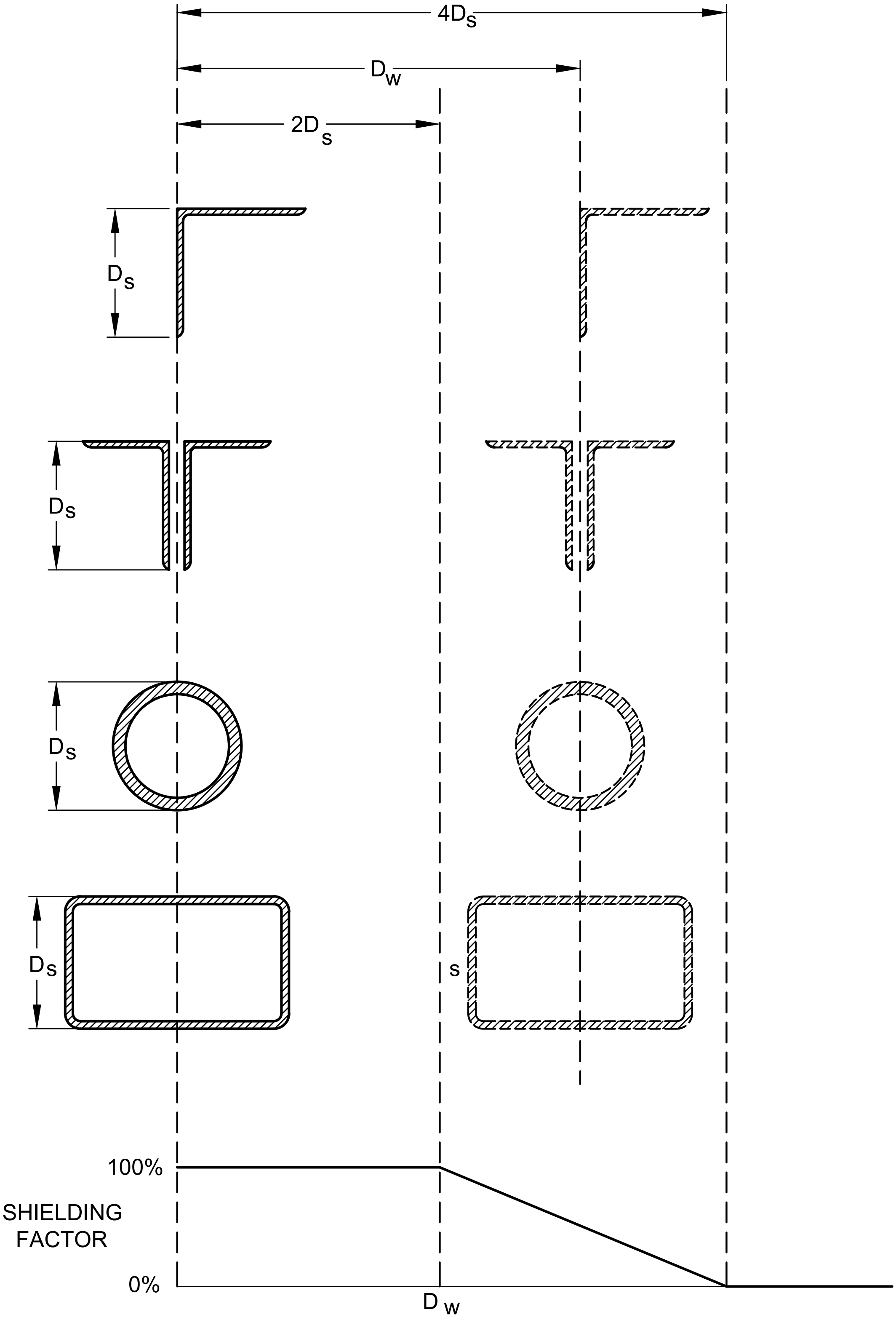


FIG. 4 SHIELDING FACTOR

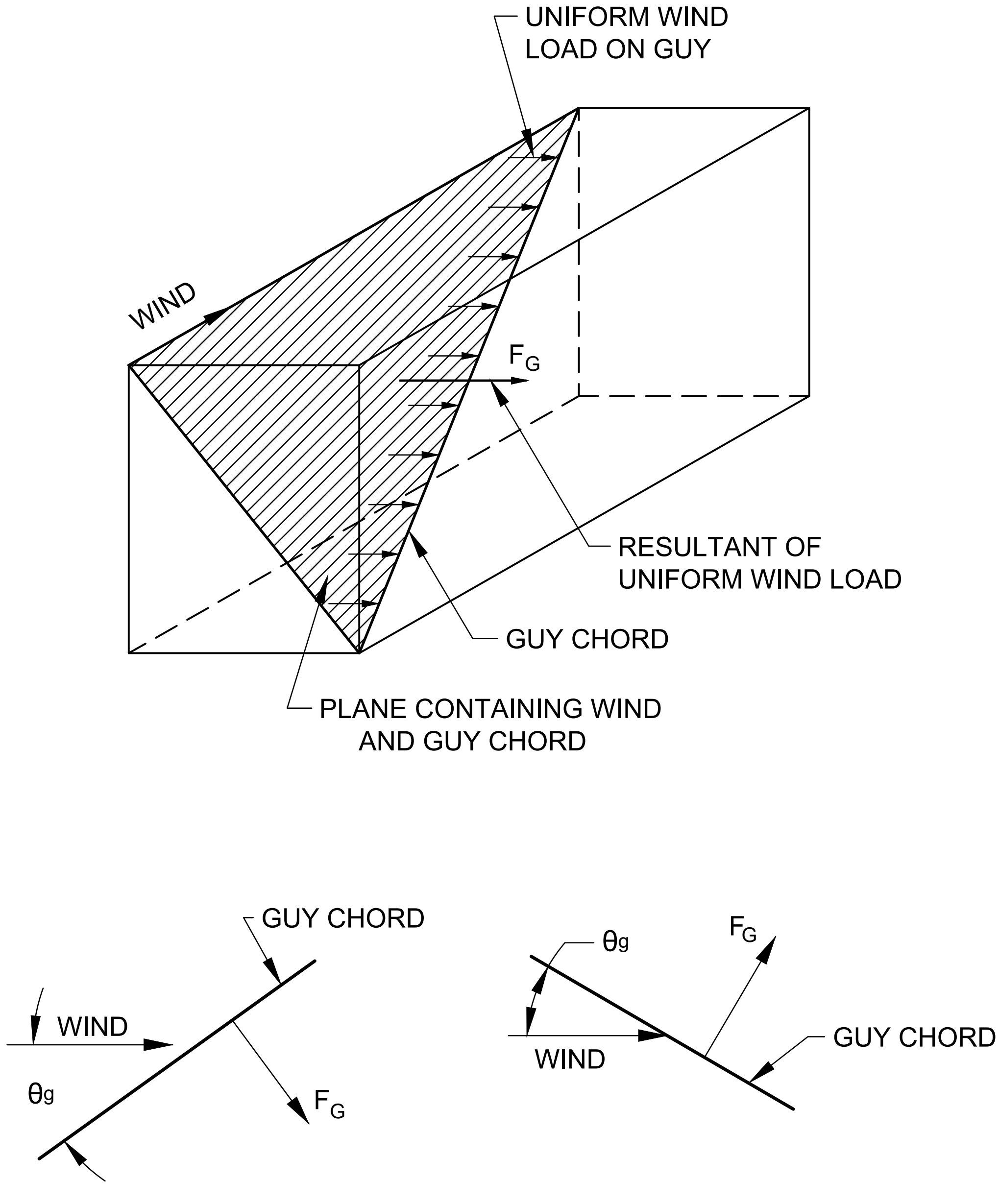


FIG. 5 WIND FORCE ON GUYS

8.4 Natural frequency, f_1 is calculated as follows for following different structures:

a) *Natural Frequency of Self–supporting Latticed Structures:*

$$f_1 = \frac{K_s\left(w_a\right)}{h^2}\sqrt{\frac{W_1}{W_1+W_2}} \text{ in Hertz}$$

where

$$W_1 = W\left[\left(\frac{w_a}{w_o}\right)^2 + 0.15\right]$$

K_s = 1500;

W = total height of the structure including appurtenances;

W_2 = weight of structure and appurtenances within top 5 percent of the structure height;

w_a = average face width of structure [m];

w_o = face width at base of structure [m]; and

h = height of structure [m].

b) *Natural Frequency of Self-supporting Pole Structures:*

$$f_1 = \frac{1}{2\,\pi}\sqrt{\frac{3E\,I_{avg}\,g}{L^3\left(W_u + 0.236\,W_L\right)}}$$

where

E = modulus of elasticity of structure material, MPa;

I_{avg} = $(I_{top} + I_{bot})/2$, mm^4;

I_{top} = moment of inertia at the top of the structure, mm^4;

I_{bot} = moment of inertia at the base of the structure, mm^4;

W_u = weight of discrete appurtenances in the top third of structure, N;

W_L = W_t-W_u, N;

W_t = total weight of the structure including appurtenances, N;

L = height of pole structure, mm; and

g = acceleration due to gravity, mm/s^2.

c) *Natural Frequency of Guyed Masts:*

$$f_1 = C_g\sqrt{\frac{K_g}{W_t}}$$

$$K_g = \sum_{i=1}^{n}\left[\frac{N_i\left(A_{gi}\right)\left(G_{ri}\right)\left(H_{gi}\right)}{h\left(L_{gi}\right)^2}\right]$$

where

C_g = 14.66;

K_g = equivalent stiffness of guys;

W_t = weight of structure including appurtenances and the total weight of all guys, Kn;

n = number of guy levels;

i = number designating guy level starting from the base to the uppermost guy level;

N_i = number of guys at guy level *i;*

A_{gi} = area of an individual guy at level *i,* mm^2;

G_{ri} = average guy radius for guys at level *i*, m;

H_{gi} = height above base to guy level *i*, m;

h = height of structure, m; and

L_{gi} = average chord length of guys at level *i*, m.

d) Alternatively, the following simplified equation may be used for the self-supporting latticed and pole structures:

$$f_1 = 50\sqrt{\frac{1}{h^{1.5}}}$$

8.5 If preliminary studies indicate that wind-induced oscillations are likely to be significant, investigations should be pursued with the aid of analytical methods or if necessary, by means of wind tunnel tests on models.

8.6 Across-wind motions may be due to lateral gustiness of the wind. Unsteady wake flow (for example, vortex shedding), negative aerodynamic damping or due to a combination of these effects. This cross-wind motion may become critical in the design of tall towers.

8.7 Motions in the direction of wind (known also as buffeting) are caused by fluctuating wind force associated with gust. The excitation depends on gust energy available at the resonant frequency.

9 ANALYSIS

9.1 Analysis Models

The structure can be modeled as the following:

a) *Self-supporting latticed towers*

1) An elastic three-dimensional truss model made up of straight members pin connected at joints, producing only axial forces in the members.

2) An elastic three-dimensional frame-truss model where continuous members (legs, K-type bracing horizontals without plan bracing) are modelled as 3-D beam elements producing both moments and axial forces in the members while the remaining bracing members which are subjected primarily to axial loads may be modelled as 3-D truss elements producing only axial forces in the members.

b) *Self-supporting pole structures*

An elastic three-dimensional beam-column model producing moments, shears and axial forces in the pole structure. Unless the analysis model considers second order effects within each element, the minimum number of beam elements shall be equal to five per pole section and the maximum beam element length shall not exceed 1.8 m

> NOTE — Due to modelling complexity (for example, meshing, element interconnection) of plate or shell models, the stresses obtained from such plate shell models shall not be less than the stresses obtained from the beam-column model noted above.

c) *Guyed masts*

1) An elastic three-dimensional beam-column where the mast is modeled as equivalent three-dimensional beam-column members supported by cables represented either as non-linear elastic supports or cable elements. This analysis produces moments, shear and axial forces in the mast, from which individual member forces may be evaluated. For finite element beam-column models, unless the analysis model considers second-order effects within each element, a minimum of five elements shall be used in any span or cantilever. The guy connection eccentric to axis of the member shall be included in the model.
2) An elastic three-dimensional truss model where individual members of the mast are modeled as straight members connected at joints producing only axial forces in the members. The cables are represented either as non-linear elastic supports or as cable elements.
3) An elastic three-dimensional frame-truss model where continuous members (legs) of the mast are modeled as 3-D beam elements producing both moments and axial forces in the members while other members may be modeled as 3-D truss members. The cables are represented either as non-linear elastic supports or as cable elements.

9.2 Application of Wind Forces to Structural Models

9.2.1 The horizontal design wind force on the structure, F_{ST}, shall be equally distributed to each leg joint of the cross-section at the panel points for three-dimensional truss or frame-truss or frame-truss models. For three-dimensional beam-column models, wind forces shall be applied as either uniform loads or as concentrated loads distributed to the nodes on the beam-column.

The horizontal design wind force, F_A, for appurtenances shall be distributed to the appropriate nodes in the middle according to the location of the appurtenance (that is, lateral load and torsion considered). For three-dimensional beam-columns, this will require applying torsional moments at the appropriate nodes.

Local bending shall be considered for structural components supporting appurtenances that are supported in the middle half of the component. For main bracing members, under this condition, local bending shall be considered for the condition of wind normal to the plane of the bracing members with no axial member load considered.

> NOTE — Weight and earthquake forces shall be distributed and considered in a similar manner.

9.2.2 *Displacement Effects*

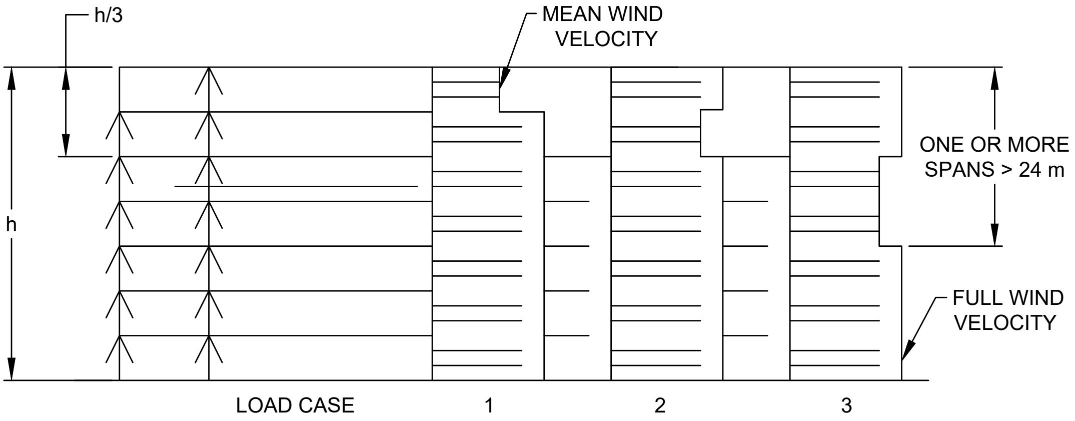
The analysis of all structures, except as provided herein, shall take into account the effects of displacements on member forces (P-Δ effects). For guyed structures the effects of displacements of the guy points as well as the effects of displacements between guy points shall be considered. For finite element beam-column models, unless the analysis model considers second order effects within each element, the minimum number of beam elements between guy levels shall be equal to five. P-Δ effects need not be considered for self-supporting latticed towers with heights less than 135 m provided that the height to face width ratios, h_i/f_{wi}, are less than 10. (h_i = Height of structure and f_{wi} = face width of structure at the level under consideration)

9.3 Wind Loading Patterns for Guyed Masts

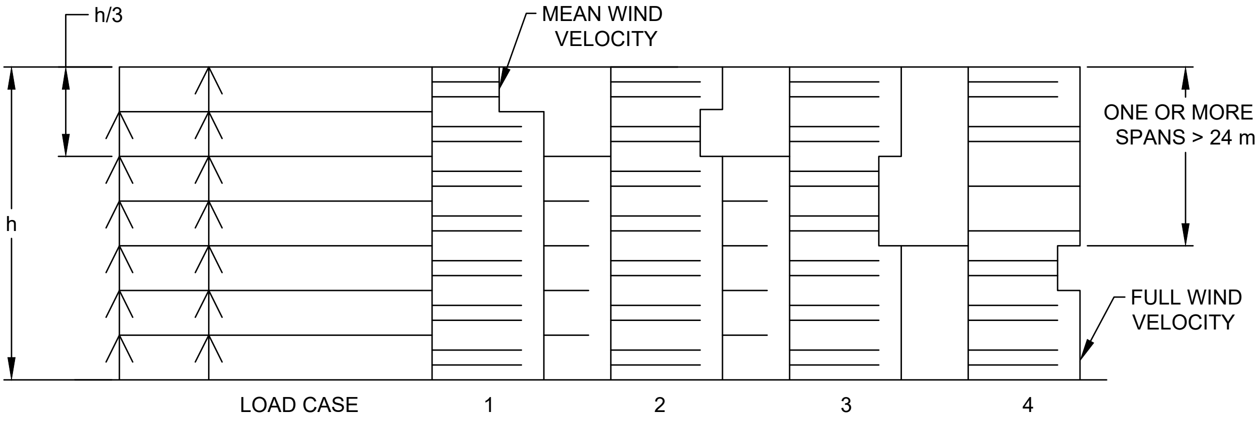
For guyed masts with three or more spans and with at least one mast span greater than 24 m within the top one-third of the height of the structure, the following wind loading patterns (*refer* Fig. 6) shall be investigated for load combination 1 as specified in **10** by varying the wind pressure as follows:

a) Wind pressure corresponding to basic wind speed over the entire height of the structure. For masts greater than 137 m in height, full wind pressure over the entire structure need not be considered when pattern loading is investigated.
b) Wind pressure corresponding to mean wind speed on the top mast span and full pressure corresponding to basic wind speed on the remaining spans.
c) Wind pressure corresponding to mean wind speed on the second mast span from the top and wind pressure corresponding to basic wind speed on the remaining spans.
d) Wind pressure corresponding to mean wind speed on the third mast span from the top and wind pressure corresponding to basic wind speed on the remaining spans.
e) Full wind pressure shall be applied on guys for all pattern loadings.

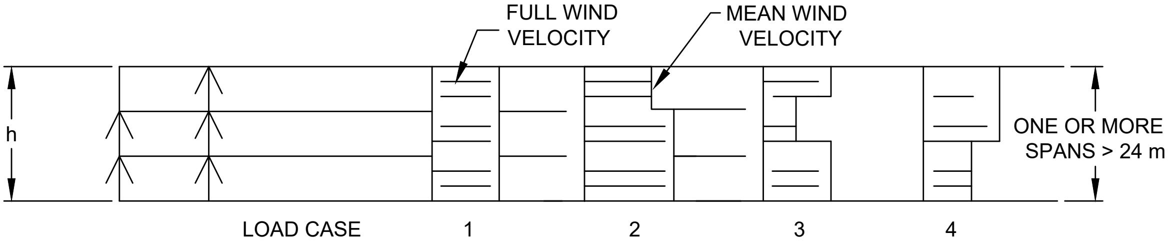
The wind pressure, $\overline{P}_d$ (design hourly mean wind pressure) is corresponding to mean wind velocity, $\overline{V}_{zd}$ and is obtained as $0.6\,\overline{V}^2_{zd}\ (\mathrm{N/m^2})$, $\overline{V}_{zd}$ = design hourly mean wind speed at height z, in m/s.



MAST HEIGHT > 137 m TOTAL LENGTH OF TOP 3 SPANS > h/3



MAST HEIGHT > 137m TOTAL LENGTH OF TOP 3 SPANS < h/3



HEIGHT LESS THAN OR EQUAL TO 135 m

NOTES

1 For masts with cantilevers (for example, broadcast antenna structures, spines, or the mast itself) the cantilevers shall be considered as the top span.

2 For masts where the total length of the top three mast spans is less than one-third of the height of the structure, the above wind loadings patterns shall be continued for each subsequent span until the total length of the considered spans is greater than one-third the height of the structure (*refer* Fig. 6).

3 When the distance between two guy elevations is less than 3 times the larger face width between the guy elevations, the wind pressure patterns shall extend to the mid-point of the two guy elevations. The short span shall not be considered as an independent span for purposes of this section.

FIG. 6 LOADING PATTERN FOR GUYED MASTS

9.4 Mast Shear and Torsion Responses for Guyed Masts

For all latticed masts, the mast face shear in a non-cantilever span due to mast shear and torsion shall not be less than 40 percent of the maximum absolute value of the face shear in the span. For all tubular masts, the mast shear in a non-cantilever span shall not be less than 40 percent of the maximum absolute value of the shear in the span.

9.5 Equivalent Beam Model for Latticed Masts

Latticed masts in a guyed structure can be replaced by series of equivalent beams. This is required for a manual analysis and it can also be done for a simplified computer analysis. The alternate is to model all the members in the mast.

The main loading on the mast having triangular, square or rectangular is the axial load P and lateral wind load, w (*see* Fig. 7). The axial load P is obtained from an overall analysis of the entire guyed structure. In addition to P and w, there might be small transverse and longitudinal end moments, as well as mid-mast moments caused by out-of-straightness including that due to bolt slippage during lifting of the mast and under full load (or test). This must be assumed and added to elastic deflection due to wind on the mast to evaluate the total lateral deflection (Δ) value. An out-of-straightness of between $L/200$ and $L/500$ is often assumed.

The mast can be analyzed as a beam-column simply supported at its ends A and B. The moments, M_0, obtained using a first order linear analysis that ignores the effects of deflection (also called primary moments) should be amplified to get the design moment, M_d, as given below:

$$M_d = M_0/(1 - P/P_{cr})$$

where P_{cr} is the buckling load of the mast. Ignoring the effect of end tapers, that can be calculated as:

$$P_{cr} = C_F \times (\pi^2 E_m I_m/h^2)$$

where

E_m, I_m = modulus of elasticity and mast moment of inertia, respectively; and

C_F = correction factor to account for the decrease the buckling strength due to deformation of bracings. A reduction of 10 percent, (for example, $C_F = 0.90$) may be used.

Moments and shears are finally resolved into individual member axial forces.

9.6 Towers with Four Guys

Approximate manual methods for calculating guy tensions can be used in towers supported by four guys. For this, assumptions are made regarding the guy or guys that are slack under a specific load case. The slack guys are simply removed from consideration.

Figure 8 shows top views of a typical four-guy patterns. The thick solid lines indicate the guys that are expected to be taut under the wind direction shown. Dotted lines indicate the slack guys that may be ignored.

10 LIMIT STATE DESIGN

To arrive at the design values of loads for the limit states of serviceability and strength the load factors given in Table 2 may be multiplied with appropriate characteristic values. Structures and foundations shall be designed so that their design strength equals or exceeds the load effects (design actions) of the factored loads in each of the limit state of strength combinations given in Table 2.

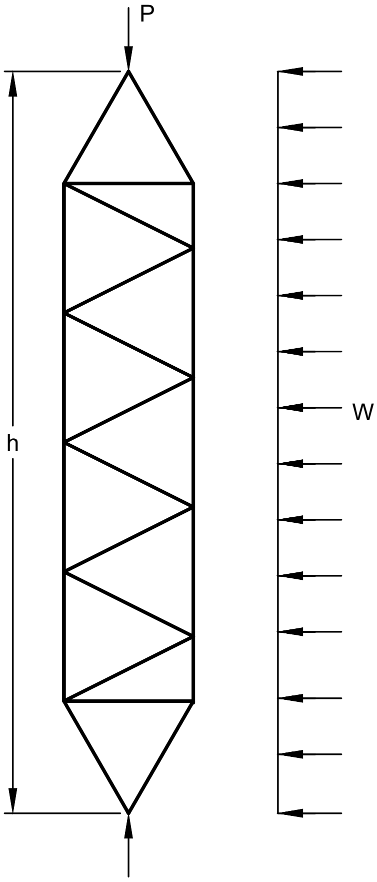


FIG. 7 GUYED MAST

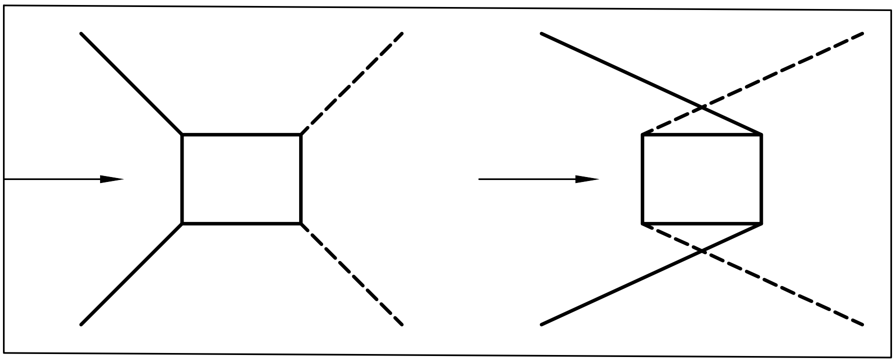


FIG. 8 TYPICAL FOUR GUYS

Table 2 Partial Safety Factors for load, γ_f for Limit States

(*Clause* 10 *and* 10.3)

Sl No.	**Type of effect on member force**	**Serviceability Load factor (For all loads)**	**Strength load factors**			
			Permanent Action[1]	Variable actions		
				Other than earthquake[2]		Earthquake
				Single[3]	More than one[4]	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Unfavourable[5]	1.0	1.1	1.4	1.1	1.0
ii)	Favourable[6]	0.0	0.9	0.0	0.0	0.0

[1] Actions, such as dead load of structure, appurtenances, guy assemblies etc.,

[2] Actions such as weight of ice due to considered ice thickness, wind load, live load, temperature effects

[3] Singly acting variable action or the most critical variable among multiple variable actions

[4] Less critical among the multiple variable actions acting simultaneously

[5] Actions that cause increase in the design forces

[6] Actions that cause a decrease in the design forces

NOTES

1 A limit state conversion factor for ice load is applied to ice thickness as per **7.2.4**.

2 Unfactored dead loads shall be used to determine earthquake loads, *E*.

3 For foundation designs, the weight of soil and substructure shall be considered as dead load in all loading combinations.

4 Exceptions:

- i) Temperature effects need not be considered for self-supporting structures.
- ii) Ice and earthquake loading need not be considered for Class I structures.
- iii) No load factor shall be applied to the initial tension of Guys.
- iv) Load combination corresponding to favourable apply to self-supporting structures only.

10.1 Limit State of Serviceability

Serviceability limit state is related to the criteria governing normal use beyond which the service criteria specified below are no longer met:

a) Deflection limit;
b) Durability consideration; and
c) Fire resistance.

The deflection under serviceability loads of the tower should not impair the performance of the structure or components the intended function. Deflections are to be evaluated for the most adverse but realistic combination of service loads and their arrangements by elastic analysis using the load factor of 1.0.

10.1.1 *Service Loads*

Service loads shall be defined by the following loading combination:

$$1.0\,D_L + 1.0\,D_g + 1.0\,W_o$$

D_L — Dead load
D_g — Dead load of guy assemblies
W_o — Wind load without ice

10.1.2 *Limit State of Deformation*

The deformation under service loads at any point on a structure intended for microwave communication, unless otherwise required, shall not exceed the following:

a) A rotation of 4 degrees about the vertical axis (twist) or any horizintal axis (sway) of the structure.
b) A horizontal displacement of 3 percent of the height of the structure.
c) For cantilevered tubular or latticed spines, poles or similar structures mounted on latticed structures, a relative horizontal displacement of 1.5 percent of the cantilever height measured between the tip of the cantilever and its base.

10.2 Minimum Thickness

The minimum thickness of galvanized and other equivalently protected tower members shall be taken as given in Table 3. Gusset plates shall be designed to resist the shear, direct and flexural stresses acting on the weakest or critical section. Re-entrant cuts shall be avoided as far as practical. Minimum thickness of gusset shall be 2 mm more than the thickness of the bracing member it connects, when the bracing member is connected to the gusset outside of the leg member. In no case, the gusset shall be less than 5 mm in thickness. Galvanized protection shall be preferred method for both members and connecting gussets, unless the painting method is proved to give an equivalent protection.

10.3 Limit State of Strength

Limit state of strength is reached when the design forces (actions) evaluated based on any load combinations with load factors given in Table 2 exceeds the design strength of the structure or its components evaluated using the partial safety factors as given below.

The ultimate strength calculation may require consideration of the following:

a) Loss of equilibrium of the structure or any part of it; and
b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of it including support and foundation.

10.4 Design Strength

10.4.1 Design strength, S_d is obtained dividing ultimate strength, S_u, by the partial safety factors for materials, γ_m, given in Table 4.

10.4.2 *Minimum Bracing Resistance*

In order to consider a reduction in the unbraced length of a supported member at a node or panel point in the plane of buckling under consideration, the bracing and secondary members shall provide a minimum resistance of 2.5 percent of axial load carried by the member. The resistance required for leg members at a panel point within a face of a tower, is given in Annex C. The minimum required design strengths of multiple members connecting at a panel point within a face can be predicted as per Annex D.

10.4.3 *Design Values of Material Strength*

For design purposes, the minimum nominal values for the yield strength and ultimate tensile strength for the type and grade of steel specified is to be used.

Table 3 Minimum Thickness of Tower Members

(*Clause* 10.2)

Sl No.	Member Type		Minimum Thickness, mm	
			Galvanized	Equivalent Protection Coating Methods
(1)	(2)		(3)	(4)
i)	Angles	Leg members and main bracing members	5	6
ii)		Other members	4	5
iii)	Hollow sections/Pipes	All members	2.6	3.2
iv)	Monopoles		3	3

Table 4 Partial Safety factors for Materials, γ_m

(*Clause* 10.4.1)

SI No.	Definition	Partial Safety Factor	
(1)	(2)	(3)	
i)	Resistance, governed by yielding, γ_{mo}	1.10	
ii)	Resistance of member to buckling, γ_{mo}	1.10	
iii)	Resistance governed by ultimate stress, γ_{m1}	1.25	
iv	Resistance for guys and their terminations	2.0	
v)	Resistance of connection	*Shop fabrication*	*Field fabrication*
	a) Bolts friction type, γ_{mf}	1.25	1.25
	b) Bolts bearing type, γ_{mb}	1.25	1.25
	c) Welds, γ_{mw}	1.25	1.50

10.4.4 *Normal Framing Eccentricities*

10.4.4.1 *Leg members*

Eccentricities at connections shall be considered unless the following conditions are met:

a) For solid round and tubular leg members, the lines of action of the bracing members meet at a point within the diameter of the leg (*see* Annex G);

b) For angle leg members, the lines of action of the bracing members meet at a point within a distance equal to 75 percent of the angle width b, on either side of the centroid of the leg angle member; and

c) For built-up leg members, the lines of action of the bracing members meet at a point with in the cross-section of the built-up leg.

10.4.4.2 *Bracing members*

The effective slenderness ratio formulas specified in **12.2** accounts for the effects of eccentric axial loading for angles connected by one leg with normal framing eccentricities. Normal framing eccentricity shall be defined as the condition where the centroid of the bolt or weld group is located between the heel of the angle and the centreline of the connected leg of the member. When a joint eccentricity exceeds this condition, the bracing member resistance shall be multiplied by the factor $b/2g$, where b is the width of the connected leg and g, is the distance from the heel of the angle to the centroid of the connection. If the width of the connected leg is 75 mm or less or the slenderness ratio, L/r, is greater than 120, the reduction factor need not be applied. Lacing bars shall be inclined at an angle not less than 40° nor more than 70° to the axis of the built-up member.

11 DESIGN OF TENSION MEMBERS

11.1 Tension Members

Tension members are linear members in which the axial forces act to cause elongation (stretch) causing essentially tensile stresses. Such members can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section (usually consisting of bolt holes). However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive plastic elongation and hence the tensile strength is limited to the gross area yield strength. Plates and other rolled sections in tension may also fail by block shear of end bolted regions. The factored design tension action T, in the members shall be less than the design tensile strength of the member, which is the lowest of the three modes of failure, as given in IS 800.

12 DESIGN OF COMPRESSION MEMBERS

12.1 Design Strength

Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by various buckling classes a, b, c and d as in IS 800.

12.1.1 *Struts*

12.1.2 *Concentric Loading* — When a single angle and other hollow sections is concentrically loaded in compression, the design strength may be evaluated using compressive design strength, as per IS 800 (*see* in Annex E).

12.1.3 *Eccentrically Loaded Members* — Strength of compression members may be evaluated using the equivalent slenderness ratio based on their end conditions.

12.2 Slenderness Ratios for Bracing Members

12.2.1 The equivalent slenderness ratio of compression and redundant members of plated sections shall be determined as given in Annex F. The effective length factors, K, for round members welded directly to leg members shall be taken from Annex F.

12.2.2 *Built-Up Members*

Individual components of built-up members composed of two or more shapes shall be connected to one another at intervals, a_i, such that the maximum slenderness ratio (a_i/r_i) of each of the component shapes between the connectors shall not exceed 100 percent of the governing effective slenderness ratio of the built-up member for composite action for the different end condition as given below:

a) A minimum of two bolts or equivalent in welding shall be provided in line across the connected width (that is, connected leg width of a double angle) at each intermittent connector point when the connected width is greater than 127 mm.

b) For buckling modes that involve relative deformations that produce shear forces in the connectors (for example, buckling about the axis parallel to the back-to-back legs for double angles), the effective slenderness ratio, $\left(\frac{KL}{r}\right)_m$, shall be modified by the following equations:

1) When either end or intermediate bolted connectors are snug-tight bearing connections and a minimum of 2 intermediate connectors are used over the length considered for out-of-plane buckling such that a_i/r_i is not greater than $0.75(KL/r)_o$:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a_i}{r_i}\right)^2} \leq \frac{KL}{r_{ib}}$$

2) When both the end and intermediate connectors are welded or bolted using high strength bolts tensioned to 70 percent of the published ultimate tensile bolt strength and the spacing of intermediate connectors result in a_i/r_i not greater than $0.75(KL/r)_o$:

$$\text{If } \frac{a_i}{r_i} \leq 40 \quad \left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_o \leq \frac{KL}{r_{ib}}$$

$$\text{If } \frac{a_i}{r_i} > 40 \quad \left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a_i}{r_i}\right)^2} \leq \frac{KL}{r_{ib}}$$

3) For all other conditions:

$$\left(\frac{KL}{r}\right)_m = \frac{KL}{r_{ib}}$$

where

$\left(\frac{KL}{r}\right)_m$ = modified effective slenderness ratio of built-up member;

$\left(\frac{KL}{r}\right)_o$ = effective slenderness ratio of built-up member acting as a unit;

r_{ib} = radius of gyration of individual component about its centroidal axis for the direction of buckling under consideration for the built-up member;

$\frac{a_i}{r_i}$ = largest slenderness ratio of individual components;

a_i = largest distance between connectors;

r_i = minimum radius of gyration of individual component; and

K_i = 0.5 for angles back to back.

= 0.75 for channels back to back

= 0.86 for all other cases

When lacing is used to connect built-up compression members consisting of two or more components, the lacing shall be triangulated and extend the full length of the member. Built-up compression members without triangulated lacing shall be modelled as a Vierendeel truss considering combined bending and axial forces. In addition, the design strength of the lacing system shall be capable of providing the resistance equal to 2.5 percentage of the connected member force.

12.3 Design of Bracings/Diagonals

The angle between any two members common to a joint of a trussed form shall preferably be greater than 20° and never less than 15° due to uncertainty of stress distribution between two closely spaced members.

It is assumed that in X-bracings system, one diagonal is subjected to compression and the other one is subjected to tension. The effective length factors are valid only if the other diagonal is subjected to a tensile force of not less than 75 percent force in the compression diagonal.

12.4 Triangular Towers

In an X-braced panel of triangular tower, one of the main diagonals (main bracings) in adjacent faces will be subjected to compression. In that case even if hip bracings are provided as in the case of square tower, the out of plane buckling of compression bracing cannot be prevented. The bracing design is governed by minor axis buckling if no elevation redundant is provided or major axis buckling if an effective elevation redundant is provided.

13 CONNECTIONS

13.1 Bolted Connections

13.1.1 *Design Strength*

The design strength of bolted connections shall be evaluated as per the provisions of IS 800.

13.1.2 *Minimum Diameter of Bolts*

The diameter of the bolt shall not be less than 12 mm. Preferred diameter of bolts used for the erection of tower, mast shall be 12, 16 and 20 mm.

13.1.3 The high strength friction grip bolts shall be tightened to 70 percent of the proof load corresponding to the bolt grade. Bearing type bolts shall be tightened to 60 to 70 percent of the proof load.

13.2 Welded Connections

Welded connections are not common in these towers. However, if they are used, the design strength of such welded connections may be evaluated as per the provisions of IS 800.

13.3 Base Plate

Base plate without gussets and with gussets shall be designed in accordance with IS 800.

13.4 Anchor Bolts

Anchor bolts shall be designed as per IS 800.

13.5 Connections in Monopoles

Monopole sections are generally connected by slip, telescopic, flange and butt joint.

14 GUY DESIGN

14.1 Guy Pretension

The pretensions of multiple guys, attached to rigid or hinged single poles of steel are of critical importance in determining the behavior of the combined structural system.

For all guyed structures the amount of pretension in the guys will have little effect on load carrying capability unless the pretension is extreme and exceeds 50 percent of the design maximum tension. Pretensions in guy wires typically range from 5 to 10 percent of the rated breaking strength (RBS). For practical considerations, the minimum pretension should be such that the leeward guys do not go slack under frequently occurring winds (say, yearly wind) or other load combinations. A tolerance of ± 10 percent of the specified pretension is considered practical during construction.

14.2 Allowable Tensions

The maximum stress limit on guys under service loads such as extreme wind, wind and ice, and extreme ice is usually limited to 65 percent of the guy's RBS, to prevent exceeding the elastic limit of the guy. This limit is to ensure that the tension does not reach the yield point of the stranding and produce permanent set, which would require re-tensioning the guys. This limit also gives a cushion or margin to the terminal devices, which frequently do not reach the RBS of the guys when installed in field.

14.3 Guy Slope

Guy slope is usually defined as the angle in degrees between horizontal and the guy or as the ratio V/H of elevation to horizontal projection. The guy effectiveness decreases for the very steeper slopes and shallower slopes. Therefore, an optimum guy slope can be found by a trial-and-error process. Guy slopes in the range of 45 to 60 degrees from horizontal are common.

14.4 Guy Anchorage

Designer should be aware of the duration of the potential guy forces. The guy loads from everyday tensions are permanent in nature and should be matched with long term strength properties of the anchors. The forces from extreme wind may last only a few seconds. For these loads of shorter duration, the anchors will normally creep less and have a greater design strength.

14.5 Guyed Poles

The guyed poles should be set in a straight vertical position. The horizontal offset tolerance should be within 75 mm. The vertical tolerance should be within 150 mm from the specified setting depth. The guy anchors can be relocated within 1 m radius of the original design location and the repositioning of the anchor should be documented and reported to the engineer for further analysis. A tolerance of 10 percent of the specified guy pretension is considered practical.

The guyed poles or masts are vertical and are capable of resisting both axial compression and bending moment.

15 FREE STANDING AND GUYED POLES

The poles may be circular or polygonal in cross section. The adequacy of the section, for combined effect of axial force and bending moment obtained from analysis, may checked as given below.

15.1 Regular Polygonal Members

The design tensile stress evaluated as given below shall not exceed f_t:

$$f_t = \frac{T}{A} + \frac{MC}{I}$$

In formed, regular polygonal tubular members, the maximum compressive stress, f_c, at the extreme fibre under combined axial force and bending moment, evaluated as given below, shall not exceed the design strength, F_a, given below:

$$f_c = \frac{P}{A} + \frac{MC}{I}$$

where,

T = Design tensile force (-ve if compressive);

P = Design compressive force;

M = Maximum design bending moment; and

A, I = Area and moment of inertia of the cross section, respectively (equations given in Annex H).

15.2 Design Strength

15.2.1 *Design Tensile strength*

The design for tensile stress should satisfy the following:

$$\frac{P}{A_g} \leq F_t \text{ where } F_t = F_y/\gamma_{m0}$$

$$\frac{P}{A_n} \leq F_t \text{ where } F_t = F_u/\gamma_{m1}$$

F_u = specified minimum ultimate Tensile strength;

F_t = Design Tensile strength;

F_y = Specified Minimum yield strength;

P = Axial tension force;

A_g = Gross cross-sectional area; and

A_n = Net cross sectional area.

15.2.2 *Design Compressive Strength*

The design for compressive stress shall satisfy the following:

a) *Octagonal, hexagonal, or rectangular members (bend angle ≥ 45°)*

$F_a = F_y'$	when $\frac{w}{t} \leq 43\,\varepsilon$
$F_a = 1.42\,F_y'(1.0 - 0.0069, \left(\frac{w}{t}\right)/\varepsilon), \leq F_y'$	when $43\,\varepsilon < \frac{w}{t} \leq 58\,\varepsilon$
$F_a = \frac{724362}{\gamma_{mo}\left(\frac{w}{t}\right)^2}, \leq F_y'$	when $\frac{w}{t} > 58\,\varepsilon$

b) *Dodecagonal members* (*12 sided members with bend angle* = 30°)

$F_a = F_y'$	when $\frac{w}{t} \leq 40\,\varepsilon$
$F_a = 1.45\,F_y'(1.0 - 0.0078, \left(\frac{w}{t}\right)/\mu), \leq F_y$	when $43\,\varepsilon < \frac{w}{t} \leq 62\,\varepsilon$
$F_a = \frac{724362}{\gamma_{mo}\left(\frac{w}{t}\right)^2}, \leq F_y'$	when $\frac{w}{t} > 62\,\varepsilon$

c) *Hexadecagonal members (sixteen sided members with bend angle = 22.5°)*

$F_a = F_y'$	when $\frac{w}{t} \leq 35.7\,\varepsilon$
$F_a = 1.42\,F_y'(1.0 - 0.00823 \left(\frac{w}{t}\right)/\varepsilon), \leq F_y'$	when $35.7\,\epsilon < \frac{w}{t} \leq 68.5\,\varepsilon$
$F_a = \frac{(724362)}{\gamma_{mo}\left(\frac{w}{t}\right)^2}, \leq F_y'$	when $\frac{w}{t} > 68.5\,\varepsilon$

where

F_y' = $F_y/\gamma_{mo;}$

F_y = specified minimum yield strength;

F_a = design compressive strength;

F_u = Specified minimum ultimate tensile strength;

t = wall thickness; and

w = flat width of a side (calculated using actual bend radius, subjected to maximum bend radius of $4t$).

$$\varepsilon = \sqrt{\frac{250}{F_y}}$$

d) *Other Sections*

1) *Rectangular Members* — The design strength specified for octagonal is used. If the design stress value exceeds 6.9 MPa, then the equations for dodecagonal members are used.

2) *Polygonal Elliptical Members* — The bend angle and flat width associated with elliptical cross section are not constant. The smallest bend angle associated with a particular flat shall be used to determine the design compressive strength.

3) *Round Members* — For round members or polygonal members with more than sixteen sides, the design compressive stress shall satisfy the following:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

where

f_a = the compressive stress due to axial load;

f_b = the compressive stress due to bending; and

F_a = design compressive strength.

$F_a = F_y'$ when $(D_o/t) \leq 104.8\,\varepsilon^2$

$F_a = 0.75\,F_y' + \frac{6555}{\frac{D_o}{t}}$ when $104.8/\varepsilon^2 \leq (D_o/t) \leq 331.2\,\varepsilon^2$

F_b is design bending strength,

$F_b = F_y'$ when $(D_o/t) \leq 165.6\,\varepsilon^2$

$F_a = 0.70\,F_y' + \frac{12420}{\frac{D_o}{t}}$ when $165.6/\varepsilon^2 \leq (D_o/t) \leq 331.2\,\varepsilon^2$

D_o is the outside diameter of the tubular section (flat to flat outside diameter for polygonal members with more than sixteen sides);

t is the wall thickness.

15.2.3 *Shear*

The shear stress resulting from applied shear forces, torsional shear, or a combination of the two shall not exceed the following:

$$\frac{VQ}{Ib} + \frac{Tc}{J} \leq F_v;$$

where

F_v = $(0.58F_y)/\gamma_{mo}$

F_y = the specified minimum yield stress;

F_v = the shear strength;

V = the shear force;

Q = the moment of section about neutral axis;

I = the moment of inertia;

T = the torsional moment;

J = the torsional moment of cross section:

c = the distance from neutral axis to extreme fibre; and

b = 2 times the wall thickness.

15.2.4 *Bending*

The stress resulting from bending shall not exceed either of the following:

$$\frac{Mc}{I} \le F_t \ or \ \frac{Mc}{I} \le F_a$$

Where F_t is the design tensile stress; F_a is the design compressive stress; I is the moment of inertia; M is the bending moment; and c is the distance from neutral axis to the extreme fibre.

15.2.5 *Combined Stresses*

For all polygonal and round members, the combined stress at critical points on the cross section shall not exceed the following:

$$\left[\left(\frac{P}{A}+\frac{M_x c_y}{I_x}+\frac{M_y c_x}{I_y}\right)^2+3\left(\frac{VQ}{Ib}+\frac{Tc}{J}\right)^2\right]^{\frac{1}{2}} \le F_t \text{ or } F_a$$

where

F_t = design tensile strength;

F_a = design compressive strength;

F_b = design bending strength;

P = axial force in the member;

A = cross sectional area;

M_x = bending moment about X-X axis;

M_y = bending moment about Y-Y axis;

I_x = moment of inertia about X-X axis;

I_y = moment of inertia about Y-Y axis;

J = torsional stiffness of the cross section;

c_x = distance from Y-Y axis to the point where stress is checked;

c_y = distance from X-X axis to the point where stress is checked;

V = shear force;

Q = moment of section about neutral axis; I is the moment of inertia:

T = torsional moment of cross section;

c = distance from neutral axis to the point where stress is checked; and

t = wall thickness.

16 FOUNDATION DESIGN

16.1 General

Foundations for freestanding towers and masts shall be designed to resist all load combinations applied to the structure in both vertical and horizontal directions combined with bending moments.

16.2 Performance of Foundation

The performance of the foundation under short term and long-term loadings shall be considered. The resulting strength and deflection shall be considered in the design of the tower. The effect of variation of water content of the soil shall be taken into account while evaluating the soil strength and shall include the allowance for reduction in the weight of materials due to buoyancy. The design of the foundation shall be carried out as per IS 11233.

16.3 Soil Properties

A soil investigation to determine the soil properties required for the design of footings shall be carried out.

16.4 Safety Factors and Design Stresses

16.4.1 Design of foundation shall be evaluated as per the provisions of IS 456, IS 4091 and IS 11233 except the provisions mentioned in these clauses.

16.4.2 While calculating the stability of foundations, the factor of safety of 1.5 shall be provided on the unfactored tower support reactions obtained from structural analysis. A factor of safety of 2.5 should be provided for safe uplift resistance in cases of piles and rock pull out anchors.

17 MAINTENANCE AND CONDITION ASSESSMENT OF COMMUNICATION TOWERS

A check list of items to be evaluated in maintenance and condition assessment are given in Annex J.

18 FIELD MAPPING OF STRUCTURES AND APPURTENANCES

Mapping of structures and components in a tower is the first step which contains all the data necessary to evaluate the loads, analysis and design of the structure and assists in fabrication and erection. Guidance for mapping of structures and components in a tower is given in Annex K.

ANNEX A

(*Clause* 2)

LIST OF INDIAN REFERENCES

IS No.	*Title*
456 : 2000	Plain and reinforced concrete (*fourth revision*)
800 : 2007	General construction in steel — Code of practice (*third revision*)
802	Use of structural steel in overhead transmission line towers
(Part 1)	Part 1 Materials, loads and design strengths
(Sec 1) : 2015	Section 1 Materials and loads (*fourth revision*)
(Sec 2) : 2016	Section 2 Design strengths (*third revision*)
(Part 2) : 1978	Part 2 Fabrication, galvanizing: inspection and packing
(Part 3) : 1978	Part 3 Testing
875	Design loads (other than earthquake) for buildings and structures — Code of practice
Part 1 : 1987	Part 1 Dead loads — Unit weights of building materials and stored materials (*second revision*)
Part 2 : 1987	Part 2 Imposed loads (*second revision*)
Part 3 : 2015	Part 3 Wind loads (*third revision*)
Part 4 : 2021	Part 4 Snow loads (*third revision*)
Part 5 : 1987	Part 5 Special loads and combinations (*second revision*)
1161 : 2014	Steel tubes for structural purposes — Specification (*fifth revision*)
1363 (Part 3) : 2018	Hexagon head bolts, screws and nuts of product grade ‘c’: Part 3 (Style 1) hexagon nuts (size range M 5 to M 64) (*fifth revision*)
1367 (Part 13) : 2020	Technical supply conditions for threaded steel fasteners Part 13 Hot dip galvanized coatings on threaded fasteners (*third revision*)
1367 (Part 6) : 2018	Technical supply conditions for threaded steel fasteners: Part 6 Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property classes — Coarse thread and fine pitch thread (*fourth revision*)
1573 : 1986	Specification for electroplated coatings of zinc on iron and steel (*second revision*)
2016 : 1967	Specification for plain washers (*first revision*)
2062 : 2011	Hot rolled medium and high tensile structural steel — Specification (*seventh revision*)
2266 : 2019	Steel wire ropes for general engineering purpose — Specification (*fifth revision*)
2315 : 1978	Specification for thimbles for wire ropes (*first revision*)
3063 : 1994	Fasteners — single coil rectangular section spring lock washers specification (*second revision*)
3757 : 1985	Specification for high strength structural bolts (*second revision*)
4091 : 1979	Code of practice for design and construction of foundations for transmission line towers and poles
4759 : 1996	Hot-dip zinc coatings on structural steel and other allied products — Specification (*third revision*)
4923 : 2017	Hollow steel sections for structural use (*third revision*)
5624 : 2021	Foundation bolts — Specification (*second revision*)
6132 (Part 2) : 1972	Shackles: Part 2 Dimensions of dee shackles
6610 : 1972	Specification for heavy washers for steel structures
6623 : 2004	High strength structural nuts (*second revision*)
6639 : 1972	Specification for hexagon bolts for steel structures
6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
8500 : 1991	Structural steel — Micro alloyed (medium and high strength qualities)

IS No.	*Title*	*IS No.*	*Title*
11233 : 1985	Code of practice for design and construction of radar antenna, microwave and TV tower foundations	12427 : 2001	Fasteners — Threaded steel fasteners — Hexagon head transmission tower bolts — Specification (*first revision*)
		IS/ISO 2415 : 2004	Forged shackles for general lifting purposes — Dee shackles and bow shackles

ANNEX B

(*Clause* 5.6)

GUYS AND APPERTENANCES

B-1 GENERAL

Guy assemblies shall be constructed of appropriate types of materials. For guy assembly components, such as preformed guy grips, mechanical sleeves, clips and other fittings, the manufacturer shall provide test results to prove the performance of the product including strength and efficiency. Guy assembly components made from ferrous materials shall be hot dip galvanized. All the design calculations shall be made for the ambient temperature of 20 degrees Celsius and adjusted in the field during erection on the basis of the prevailing temperature.

B-2 WIRE ROPE AND WIRE STRAND

B-2.1 The guys are typically of strand construction which permits sufficient flexibility for installation. Greater individual wire sizes give better corrosion resistance and higher effective modulus of the complete stranding than do wire ropes. Typical strand configuration is shown in Fig. 9. The wire ropes are made of multiple strands but are not normally used in guyed structures. These ropes are more flexible as they are made of smaller individual wires. They are also more susceptible to corrosion and the effective modulus of elasticity of the rope is lowered in an almost inverse relationship to the flexibility.

Galvanized high-strength strands are most common. The tensile (rope grade) designated are 1570, 1770 and 1960. The type of ropes shall be round, oval and flattened. The oval type is restricted to sizes from 8 to 40 mm. The flattened strand is for special applications and ranges from 13 to 48 mm sizes. The different types of steel wire ropes, their mass and breaking strengths are given in IS 2266. The typical wire rope construction of 6 × 7 (6-1) is shown in Fig. 10 pre-formed wire ropes are generally used as they have superior flexibility and have better bending resistance.

B-2.2 Protection of Wires and Ropes

The wire ropes unless properly protected against oxidation tend to corrode fast and result in fatigue failures. Therefore, the wires and the ropes are galvanized unless otherwise demanded. In order to reduce friction, wire ropes are well lubricated. The lubrication requirement is dependent on the application of the wire rope. In case of ropes for general engineering purposes, the grease used shall be free from solvents, acidity, alkalinity, moisture or any other aromatic compound.

B-3 SHACKLES AND TURNBUCKLES

Guyed structures have traditionally been erected with guys attached at their tops and with temporary fittings and tackle to the anchors while the structure is made almost plumb. Permanent grips are installed and final positioning and pre-tensioning done with some form of threaded device. These can be simple threaded extensions of anchor rods, turnbuckles which can be very expensive in the larger sizes, or threaded U-bolts with keepers.

Shackles and turn buckles shall be forged steel or equivalent material and suitably heat treated [*see* IS/ISO 2415, IS 6132 (Part 2)].

B-3.1 Take-up Devices

Turnbuckles or other suitable means of length adjustment that do not necessitate disconnecting end attachments shall be provided for all guys.

B-3.2 Thimbles

A thimble or other means shall be used to maintain as adequate radius at the inside of the end attachments consisting of loops. Thimbles shall be of the heavy duty or solid type and the size shall be in accordance with IS 2315 or with the manufacturers' recommendations.

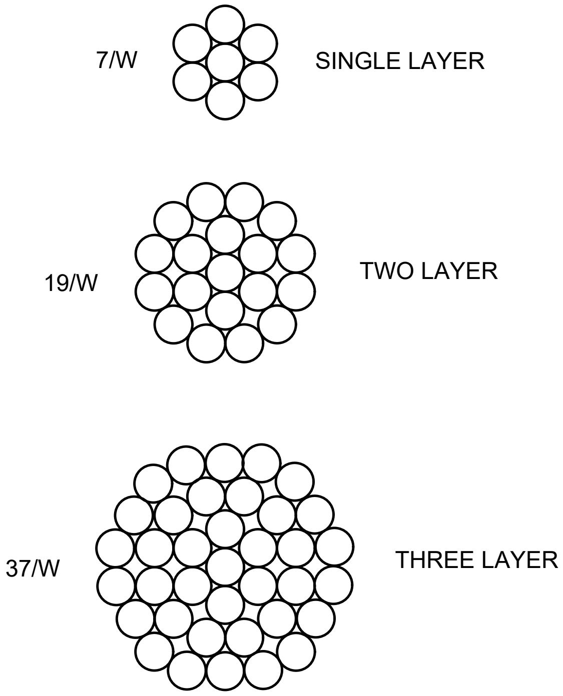
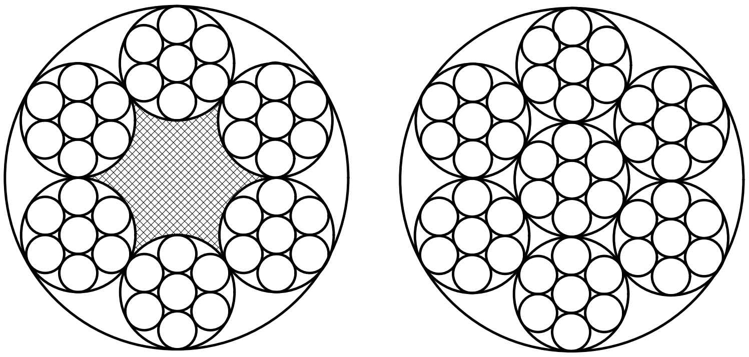


Fig. 9 Strand Configuration


WITH FIBER CORE (CF)

WITH STEEL CORE (CWR)

Fig. 10 Wire Rope Construction of 6 × 7 (6-1)

B-3.3 Preformed Guy Grips

Preformed guy grips shall be designed specifically for the type of the guy being used. This includes electrochemical compatibility of the material and the number, size, and lay of wires in the strand.

B-3.4 Mechanical or Pressed Sleeves

Loops may be secured by means of mechanical type splice consisting of metal ferrule made cold under heavy pressure. The ferrule shall be of rustproof material, electrochemically compatible with the rope being spliced. Care shall be taken in the application of the sleeves to ensure that the individual wires of the strand or rope being spliced are not broken or crushed.

B-3.5 Clips

Twin- base or U-bolt wire rope clips may be used to secure looped ends. The clips shall be of the same size as the guys within tolerance of ± 1.2 mm. Clips shall be installed in accordance with the manufacturer's instructions regarding number, spacing and tensioning.

B-3.6 Sockets

Sockets shall be cast such that they transmit the full load of the guy without permanent deformation of the socket and they shall be heat treated.

B-3.7 Guy Link Plates

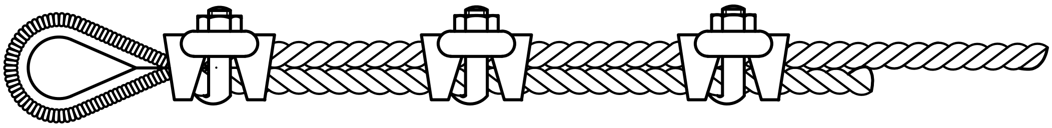
Where guy link plates are used, two plates shall be parallel and connected so that there is no relative moment of the plates. Strand or wire ropes shall not be spliced by means of a running or woven type of splice.

B-3.8 Guy Fittings

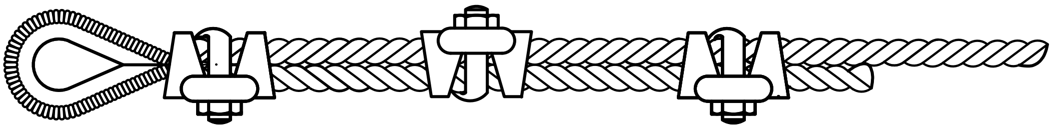
There are many different and suitable termination devices (Fig. 11) for the smaller and relatively flexible strand sizes up to 12 mm diameter. Bolted clamps, fittings with wedges, preformed grips, and countless other types have been used with success and the selection depends on local availability and confidence in long-term holding strength.

The overall rigidity of strands above 12 mm diameter precludes devices, such as the three-bolt clamp that must deform the strand to perform. Clamps that depend on surface friction can become very lengthy on large strands as the needed holding strength varies as the diameter squared whereas the surface area varies only with the diameter.

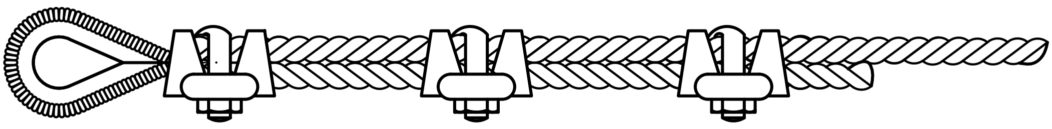
Standard preformed grips or helical wire terminals perform well up to about 25 mm stranding diameter. Beyond that point, special long grips are required or the twisted grip part may have to be supplemented by a cone wedge device. Preformed grips may be reapplied



CORRECT METHOD



INCORRECT (CLIP STAGGERED)



INCORRECT (CLIP REVERSED)

FIG. 11 THIMBLE OF GUY ROPE AND POSITION OF CLAMPS

up to two times for the purpose of re-tensioning with no significant loss of holding strength, but if installed for more than about three months, they should be replaced after removal. The apparent and actual ease of installation and removal sometimes at tracts vandalism which can be thwarted by compressing a ring clip to the upper end of the assembly.

Swaged or compressed fittings are economical and easily applied in the shop or field but care must be taken in the selection of the steel. Some compressed fittings have cracked in brittle fracture under a combination of extremely low temperature and high internal stress levels that are produced during the swaging operation. This problem is no different than that of correct steel selection for the dead-end fittings for the conductor although larger stronger fittings are sometimes required for the guys.

B-3.9 Cable Behaviour

The cable behaviour is most often modelled similar to that of straight bars of similar mechanical properties. It is important that the designer of guyed structures understand conditions under which the straight bar model is applicable.

The relationship between tension and elongation is assumed linear; that is, the guy effective modulus of elasticity E is assumed constant. If the guy cable were a straight steel bar, its modulus of elasticity would be that of steel, E_s = 200 000 MPa. However, because a guy is normally made of wound strands, its effective modulus of elasticity E is smaller than E_s. A value of 160 000 MPa is often used.

B-4 GUY INSTALLATION

The top ends of the guys have a permanent nonadjustable dead-end fitting and these are attached to the masts before erection of the structure. The remainder of the operation of attaching the guys to the anchors, the plumbing of the structure, and then the pre-tensioning of the whole system can be done by at least two different methods.

B-4.1 Traditional Method

The guys are cut to approximate length (but always slightly longer than needed), permanently attached at their tops to the structure, and then attached at their bottoms with temporary grips or other wedge-type devices. The structure is held in approximate plumb position by crane or helicopter while some block and tackle or tensioning device takes up the slack in each guy and holds the structure in place. Surveying equipment and plumb bobs are used while the structure is brought into proper alignment by tightening and slackening the guys; the loads are then transferred to the permanent fitting which contains threaded tensioning devices used to apply the specified pretension. This operation can involve many person-hours, especially if close and rigid specifications have been set regarding the positioning of the tower and the pretension levels.

B-4.2 Alternate Method

This method is advantageous if one accepts the fact that relaxed or more liberal than usual tolerances can be applied to the erection of guyed rigid frames and masted towers. These relaxed tolerances often will have negligible effect on the strength or performance of the structures, the tolerances on erection, and positioning of the structure being limited to what is needed to ensure that the structure does not appear other than vertical.

The human eye cannot detect mast or tower is out of plumb by less than about one degree. One-half of a degree has been accepted by some as a limit. Furthermore, the tensions in guys, in the absence of wind, are all equal and any pre-tensioning can be introduced by using a threaded tensioning device in the guys. A survey is made of the elevations of the tops of the anchors and of the mast footing and calculations made of the needed guy lengths. Guys are then cut and fixed with simple dead-end fittings; an adjustable device added to one only. Upon erection of the structure, it is moved slightly at the top to permit direct attachment of the guys to their anchors and final one is attached and the desired pretension introduced. The alternate method requires that the lengths of the guys and their fitting be accurate to about 0.3 percent of calculated length, which is a reasonable degree of precision.

B-5 GUYED MASTS

B-5.1 The following geometrical and mechanical properties are used to characterize a guy cable.

h = structure height to lower guy (m);

L = chord length-from anchor to structure attachment point (m);

α = guy slope-angle measured from horizontal to chord (degree);

A = horizontal displacement of structure attachment point (mm);

D = outside diameter (mm);

A = cross-section area (mm)^2;

W = bare weight per unit length (N/m);

RBS = rated breaking strength or ultimate tension capacity (kN);

Fe = tension force in guy-assumed constant along guy (kN);

HF_c = horizontal component of tension force in guy (kN);

VF_c = vertical component of tension force in guy (kN);

Pf_c = pretension force-value of tension in unloaded structure expressed as a fraction of RBS (percent);

E = guy effective modulus of elasticity (MPa); and

K_c = horizontal guy stiffness-horizontal guy reaction HFc caused by a unit horizontal displacement Δ (kN/m).

B-5.2 Single Guy Level and Hinged Base

Figure 12 shows a guyed pole with a single back guy subjected to a transverse force H. Because the pole is hinged at the base, the entire transverse force is resisted by the horizontal component of the guy tension; that is, $HF_c = H$. The vertical component VF_c follows from the guy slope. That component creates compression in the pole:

$VF_p = VF_c$.

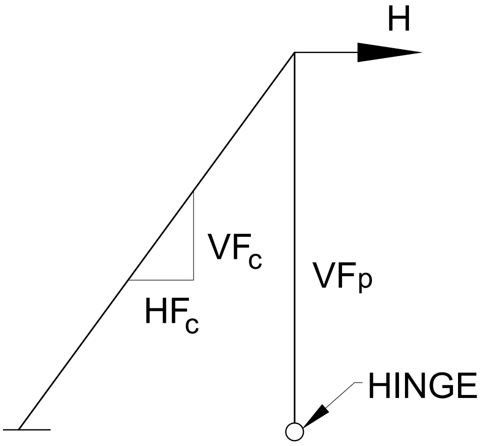


FIG. 12 GUYED MAST

In Fig. 13, the transverse force is resisted by a pair of back guys. The top view of the system shows how each guy shares the transverse load equally. The force in each guy can be obtained by similar prisms: $F_e = (H/2)\ L/t$, where t, l, and h are the transverse, longitudinal, and vertical projections of the guy, and L is the guy length.

In Fig. 14, the pole is held by a pair of 45° prestressed guys, a back guy AC, and a head guy BC. Assume that the guys are prestressed at 14.14 kN. This pretension causes an axial force of 20 kN in the pole. When the transverse load is increased from zero to 30 kN, the tension in the back guy increases from 14.14 to 42.43 kN, whereas that in the head guy decreases from 14.1 kN to zero. The head guy becomes slack for loads larger than or equal to 20 kN. For a load of 30 kN, the back guy tension is 42.43 kN, exactly the same as if there had been no prestress. From this it may be noted that as long as a guy prestress does not exceed 50 percent of the maximum loaded guy tension, the prestress will not have an effect on the maximum tension. Before the head guy becomes slack, the guy tensions are simply calculated as the superposition of the initial guy preloads and the forces in the truss made up of the mast and two straight bars replacing the guys.

When a hinge exists or is assumed to exist at the base of a pole, the corresponding model is referred to herein as a 'column or strut' model and the analysis as a 'column or strut' analysis.

B-5.3 Single Guy Level and Fixed Base

Figure 15 shows a pole fixed at its base and supported by a single back guy. The system is now statically indeterminate. The transverse load H is shared between the horizontal component of guy tension HF_G and the shear in the pole HF_P; that is, $H = HF_G + HF_P$. The relative magnitudes of the forces HF_G and HF_P depend on the horizontal stiffness's K_G and K_P of the guy and the pole, respectively. The situation is similar to that of two springs sharing the load. It can easily be shown that:

$$HF_G = H \times K_G/(K_G + K_P);\text{ and}$$

$$HF_P = H \times K_P/(K_G + K_P)$$

The ratios $K_G/(K_G + K_P)$ and $K_P/(K_G + K_P)$ are distribution factors distributing the transverse load to the two components of the system. The moment at the base of the pole is:

$$M = h \times HF_P.$$

For a constant cross-section pole of height h, modulus of elasticity E_P, and moment of inertia I_P**:**

$$K_P = 3 \times E_P \times I_P / h^3$$

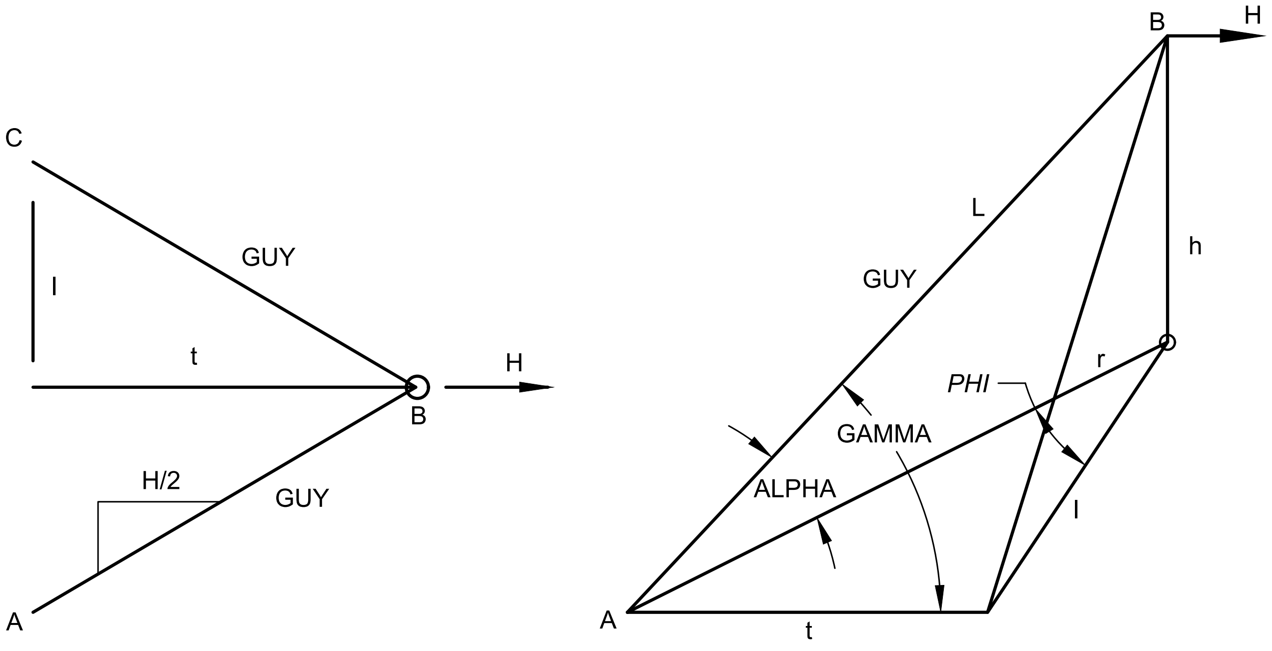
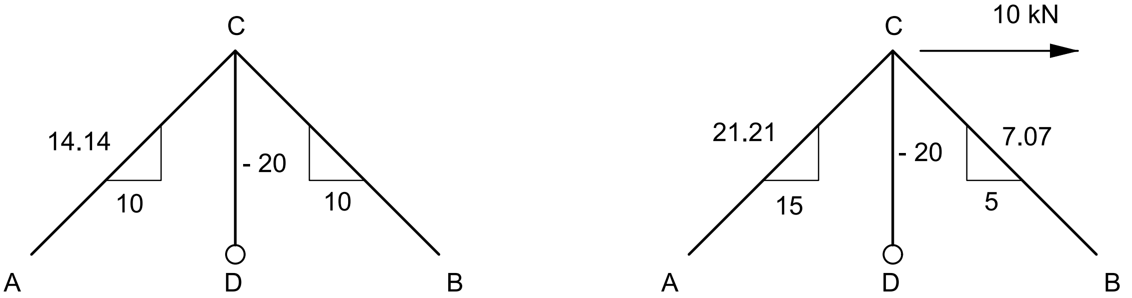
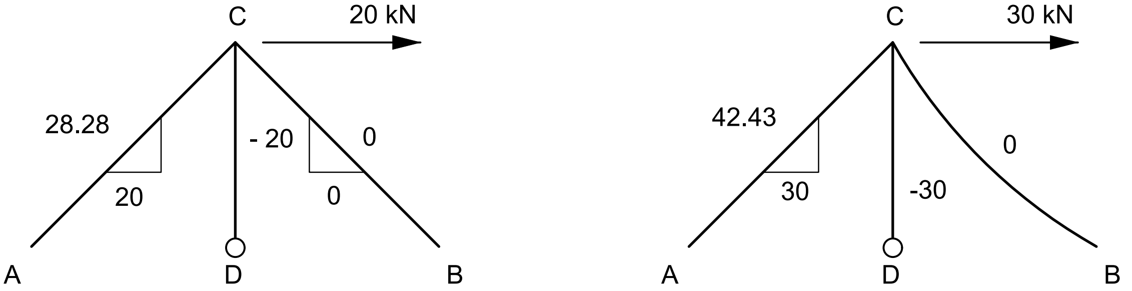


Fig. 13 Hinged Mast with Two Guys



PRETENSION

10 kN TRANSVERSE



20 kN TRANSVERSE

30 kN TRANSVERSE

Fig. 14 Hinged Mast with Inline Back and Front Guys

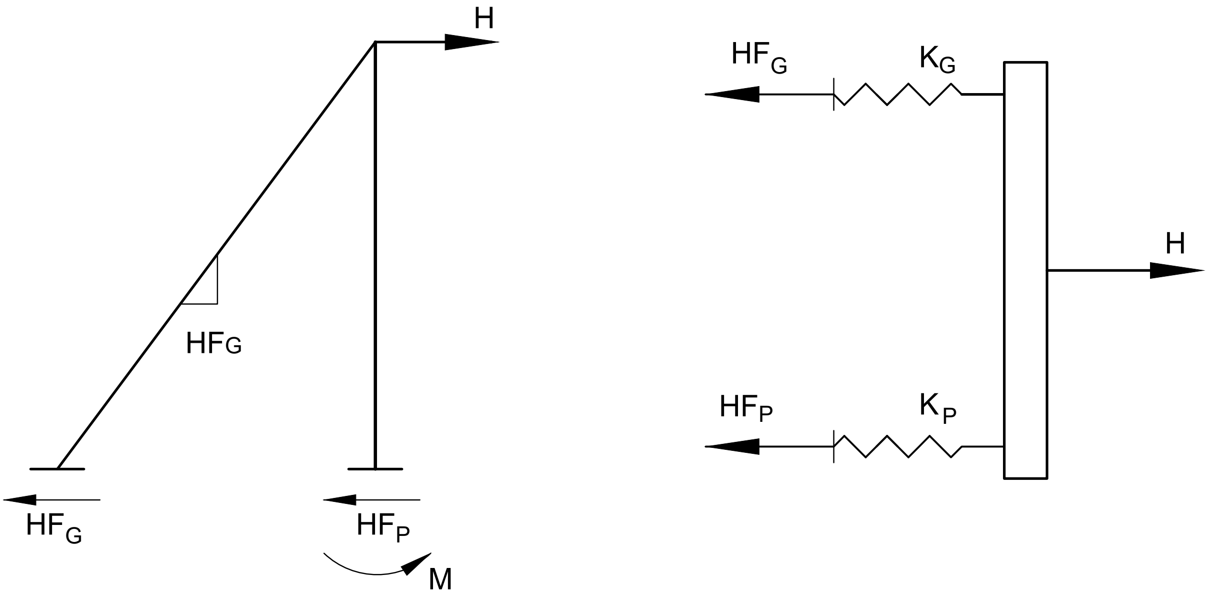
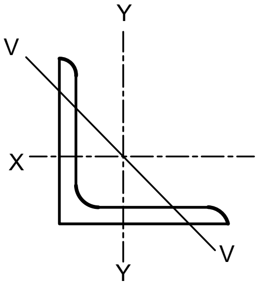
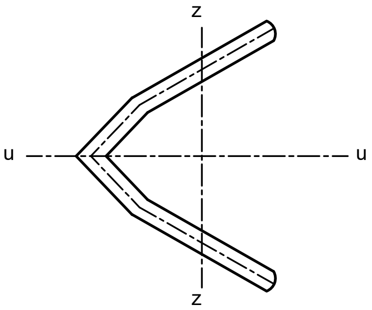
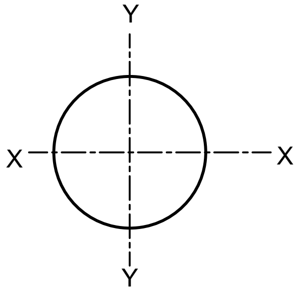


Fig. 15 Fixed Base Pole with In-Line Guy

ANNEX C

(*Clause* 10.4.2)

BRACING RESISTANCE REQUIRED FOR LEG MEMBER

Leg Shape	Tower Cross Section	Bracing Resistance, P_r Required in a face at a Panel Point
	Square	When weak-axis buckling (KL/r_v) governs: $P_r = P_s/(2 \times 0.707) = 0.707\ P_s$ When in-Plane buckling (KL/r_x or KL/r_y) governs: $P_r = P_s$
	Triangular	When weak-axis buckling (KL/r_z) governs: $P_r = P_s/(2 \times 0.866) = 0.577\ P_s$ When in-plane buckling (KL/r_u) governs: $P_r = P_s$ When in-plane buckling (KL/r_u) governs: $P_r = P_s/(0.866) = 1.15\ P_s$
 TUBULAR OR SOLID	Square	When in-Plane buckling (KL/r_x or KL/r_y) governs: $P_r = P_s$
	Triangular	Out-of-plane buckling (KL/r_x) governs: $P_r = P_s/(0.866) = 1.15\ P_s$

where

$$P_s = \left[1.5 + \frac{\left(\frac{KL}{r} - 60 \right)}{60} \right] \frac{F_s}{100} \qquad 1.5\frac{F_s}{100} \le P_s \le 2.5\frac{F_s}{100}$$

P_r = Resistance required to provide lateral support at a panel point within a face of a latticed structure;

P_s = Minimum bracing resistance normal to a supported member required to provide lateral support;

NOTES

1 Alternatively, P_r may be determined using the worst case effective slenderness ratio (highest ratio) to determine P_s and multiplying the result by 1.15 for triangular cross sections or by 1.00 for square cross sections.

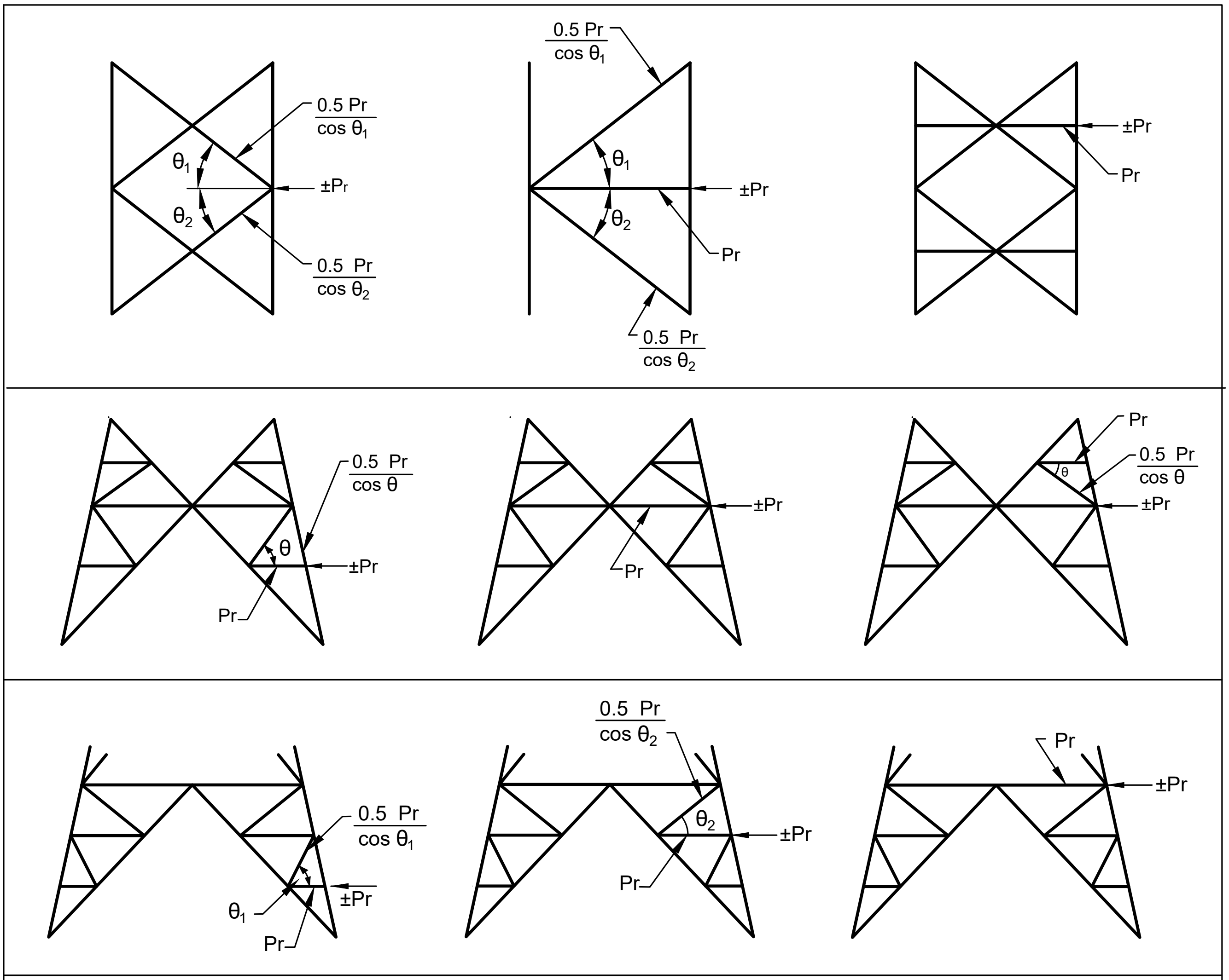
2 One value of P_r applies for both faces when investigating a segment of a leg. The larger value shall be used considering the leg segment above and below a panel point.

3 Weak-axis buckling governs angle legs for symmetrical bracing patterns. In-plane, out-of-plane or weak-axis buckling may govern staggered bracing patterns.

ANNEX D

(*Clause* 10.4.2)

MINIMUM REQUIRED RESISTANCE AT PANEL POINTS



NOTE - For leg slopes greater than 15 degrees from vertical, P_r shall be divided by the Cosine of the leg slopes

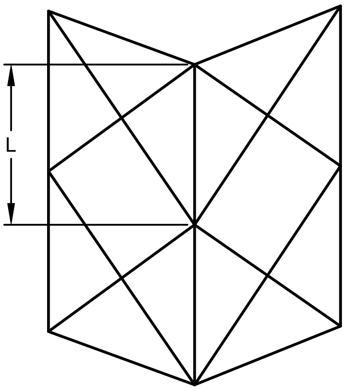
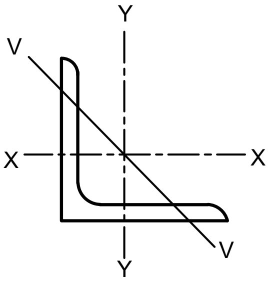
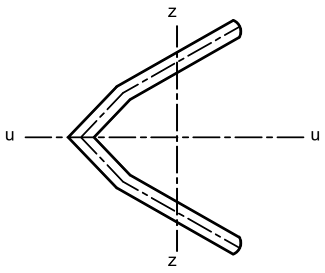
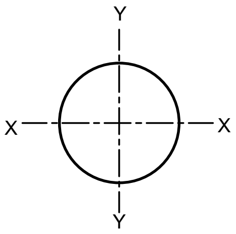
ANNEX E

(*Clause* 12.1.2)

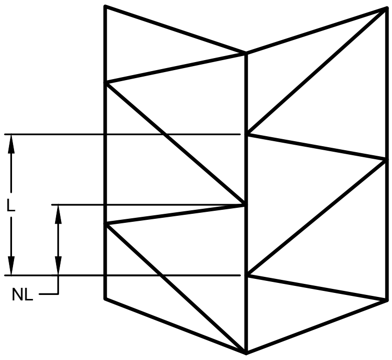
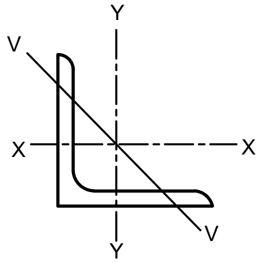
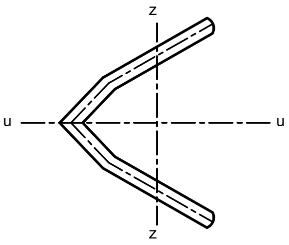
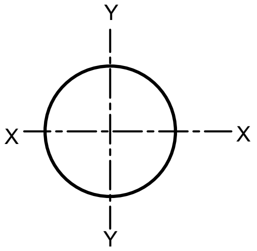
EFFECTIVE SLENDERNESS RATIO FOR LEG MEMBERS

E-1 EFFECTIVE SLENDERNESS RATIO OF LEG MEMBERS OF BRACING SYSTEMS

E-1.1 Symmetrical Bracing System

	Leg Shapes	***Effective Slenderness Ratios K = 1.0***
		KL/r_v
		KL/r_z
	 TUBULAR OR SOLID	KL/r_x

E-1.2 Staggered Bracing Patterns

	Leg Shapes	***Effective Slenderness Ratios K=1.0***
 $N \geq 0.5$		KL/r_x, KL/r_y, $[(1+2N)/3]KL/r_v$
		KL/r_z, KL/r_u, $[(1 + 2N)/3]\ KL/r_z$, $[(1 + 2N)/3]\ KL/r_{tf}$ Where, r_{tf} = Equivalent radius of gyration governing torsional flexural buckling
	 TUBULAR OR SOLID	KL/r_x

NOTES

1 L shall equal the panel spacing measured along the axis of the leg.

2 The maximum effective slenderness ratio shall be used to determine the design compression strength and the bracing resistance required to provide lateral support.

E-2 EFFECTIVE SLENDERNESS RATIO OF DIFFERENT ANGLE LEG SHAPES

<table>
<tr><th>Leg Shapes</th><th>Effective Slenderness Ratios K=1.0</th></tr>
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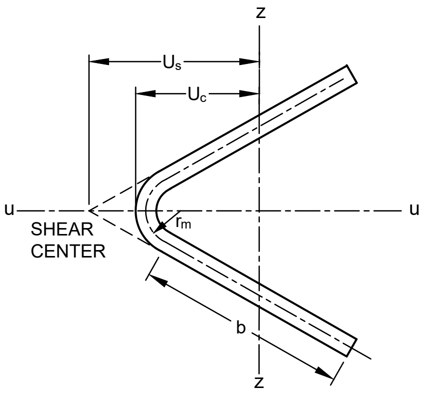

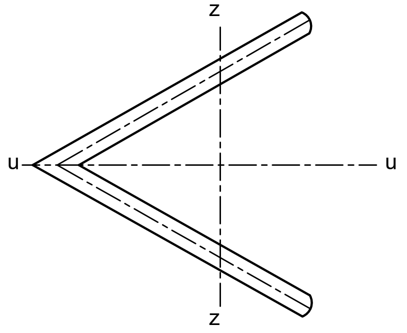

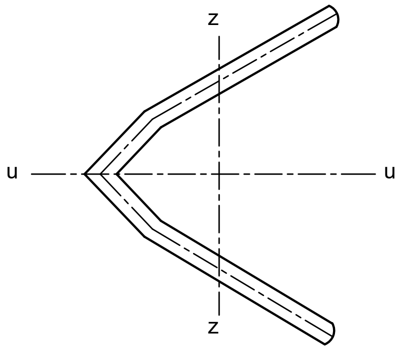

</td><td>

The design stress shall be computed for the larger of KL/r_z and KL/r_{tf} given by

$$\frac{2}{r_{tf}^2}=\frac{1}{r_t^2}+\frac{1}{r_u^2}+\sqrt{\left(\frac{1}{r_t^2}-\frac{1}{r_u^2}\right)^2+4\left(\frac{u_0}{r_t r_u r_{ps}}\right)^2}$$

$$r_t=\sqrt{C_w+\frac{0.04J(K_tL)^2}{I_{ps}}}$$

where

$C_w =$ Warping Constant:
$J =$ St. Venant’s torsion Constant:
$K_t =$ Effective length coefficient for warping restraint:
$L =$ Unbraced Length of member:
$r_u =$ Radius of gyration about u-axis:
$u_0 =$ Distance between shear centre and centroid:
$r_{ps} = \sqrt{\frac{I_{ps}}{A}}$ Polar radius of gyration about shear centre,
$I_{ps} = I_u + I_z + Au_0^2 =$ Polar moment of inertia about shear center:
$I_u =$ Moment of inertia about u-axis:
$I_z =$ Moment of inertia about z-axis:
$A =$ Area of cross section.
$K_t =$ 1

$KL/r_z, KL/r_{tf}$

$r_{tf} =$ Varies with length
$r_{tf} =$ equivalent radius of gyration

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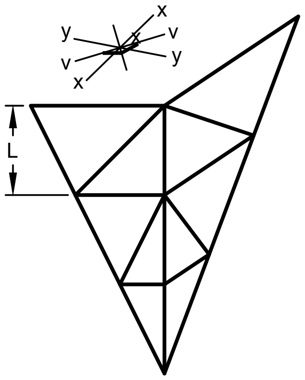
</table>

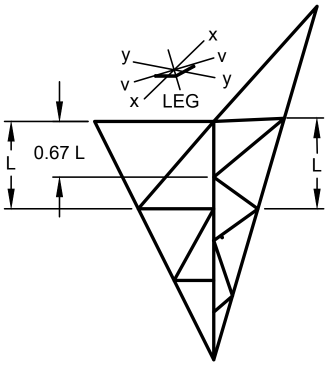
E-3 EFFECTIVE SLENDERNESS RATIO OF LEG MEMBER OF DIFFERENT BRACING SYSTEMS

LEG MEMBER USING SYMMETRICAL BRACING



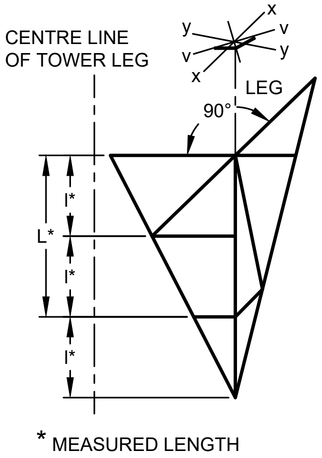
Method of loading/Rigidity of Joints	Slenderness ratio
Concentric Loading	L/r_{vv} from 0 to 120 $(KL/r = L/r_{vv})$
No restraint at ends	L/r_{vv} from 120 to 200 $(KL/r = L/r_{vv})$

LEG MEMBER USING STAGGERING BRACING



Method of loading/Rigidity of Joints	Slenderness ratio
Concentric Loading	L/r_{xx} or L/r_{yy} or $0.67\ L/r_{vv}$ from 0 to 120 $(KL/r = L/r)$
No restraint at ends	L/r_{xx} or L/r_{yy} or $0.67\ L/r_{vv}$ from 120 to 200 $(KL/r = L/r)$

LEG MEMBER USING STAGGERING BRACING (NUMBER OF STAGGERED PARTS UPTO 3)



Method of loading/Rigidity of Joints	Slenderness ratio
Concentric Loading	L/r_{xx} or L/r_{yy} or L/r_{vv} from 0 to 120 $(KL/r = L/r)$
No restraint at ends	L/r_{xx} or L/r_{yy} or L/r_{vv} from 120 to 200 $(KL/r = L/r)$

NOTE — If total number of staggered parts up to 3 as shown in figure

E-4 EQUIVALENT SLENDERNESS RATIOS BASED ON END CONDITIONS

E-4.1 The equivalent slenderness ratios of leg members, based on end conditions and types of members, are given at Table 5.

Table 5 Equivalent Slenderness Ratios Based on End Conditions

(*Clause* E-4.1)

Sl No.	Type of Member and End Conditions	Value KL/r
(1)	(2)	(3)
Concentric at both ends (for example, double angles or channels, round members with concentric end plates)		
i)	Leg sections or members bolted on either ends in both flanges	
ii)	Member with concentric loading at both ends of the unsupported panel with value of ***L*/*r*** up to and including 120	***L*/*r***

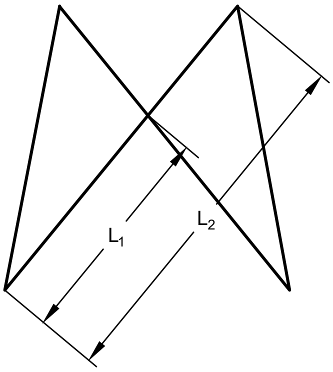
NOTE
Limiting value of *KL/r of* Leg members = 150

ANNEX F

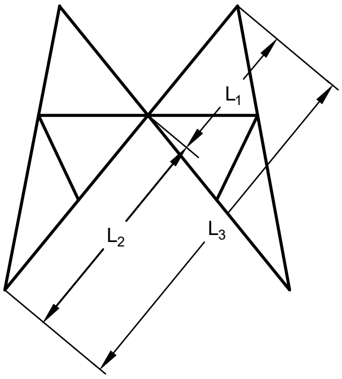
(*Clause* 12.2.1)

EFFECTIVE SLENDERNESS RATIO OF BRACINGS

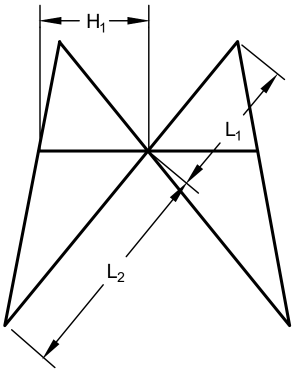
F-1 X-BRACING SYSTEM WITH AND WITHOUT SECONDARY BRACINGS



Cross over point Providing support	Cross over point Connected but no support provided
L_1/r_{min}	L_1/r_{min} or L_2/r_{max}



Cross over point Providing support	Cross over point Connected but no support provided
L_1/r_{min} L_2/r_{max}	L_1/r_{min} L_3/r_{max}



Internal bracing at Cross over point	No internal bracing at Cross over point
L_1/r_{min} L_2/r_{min} H_1/r_{min}	L_1/r_{min} L_2/r_{min} H_1/r_{min} $2H_1/r_{max}$

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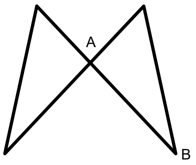
<td>
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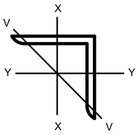
<td>
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<td>AB/ r_{vv}</td>
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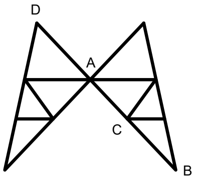
<td>
</td>
<td></td>
<td>AC/r_{vv} or CB/r_{vv}
$*AB/r_{xx}$ or $*AB/r_{yy}$ or $*AD/r_{vv}$</td>
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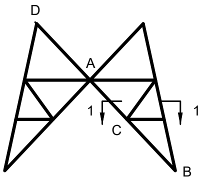
<td>
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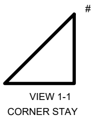
<td>
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<td>AC/r_{vv} or CB/r_{vv}
$*AD/r_{vv}$</td>
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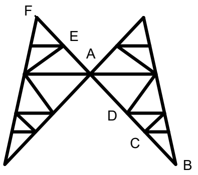
<td>
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<td></td>
<td>AD/r_{vv} or $*AF/r_{xx}$
DC/r_{vv} or AE/r_{vv}
CB/r_{vv}
$*AB/r_{xx}$ or $*AB/r_{yy}$</td>
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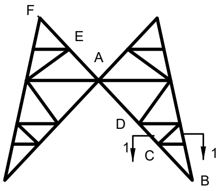
<td>
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<td></td>
<td>AD/r_{vv} or $*AF /r_{xx}$
DC/r_{vv} or AE/r_{vv}
CB/r_{vv}
$*AC/r_{yy}$</td>
</tr>
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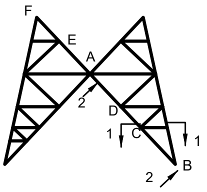
<td>
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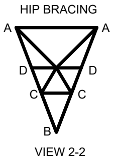
<td>
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<td>AD/r_{vv} or $*AF/r_{xx}$
DC/r_{vv} or CB/r_{vv}
$*AE/r_{vv}$</td>
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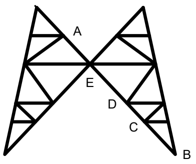
<td>
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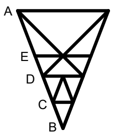
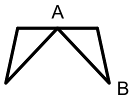
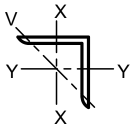
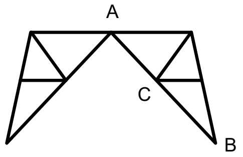
<td>
</td>
<td>AE/r_{vv} or $*AF/r_{xx}$
ED/r_{vv} or $*AE/r_{vv}$
DC/r_{vv} or CB/r_{vv}</td>
</tr>
<tr>
<td colspan="3">X- BRACINGS WITH AND WITHOUT SECONDARY MEMBERS</td>
</tr>
<tr>
<td colspan="3">* Application for tension compression system only i.e tensile stresses in one bracing must be atleast equal to 75 percent of the compressive stress in other bracing
The corner stay should be designed to provide lateral support adequately</td>
</tr>
</table>

F-2 K-BRACING SYSTEM WITH AND WITHOUT SECONDARY BRACINGS



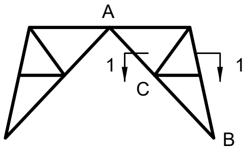


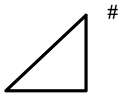
AB/ r_{vv}



AC/r_{vv} or CB/r_{vv}

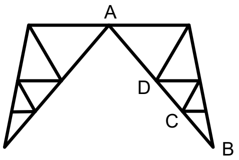
AB/r_{xx} AB/r_{yy}



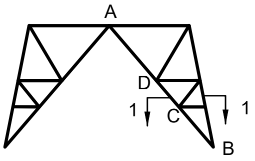


VIEW 1-1

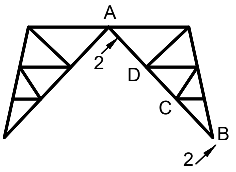
AC/r_{vv} or CB/r_{vv}



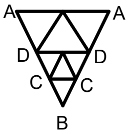
AD/r_{vv}
DC/r_{vv}
CB/r_{vv}
AB/r_{xx} or AB/r_{yy}



AD/r_{vv}
DC/r_{vv}
CB/r_{vv}
AC/r_{xx} or AC/r_{yy}

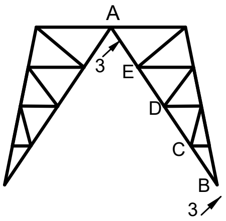


HIP BRACING

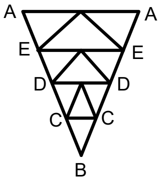


VIEW 2-2

AD/r_{vv}
DC/r_{vv}
CB/r_{vv}



HIP BRACING



VIEW 3 - 3

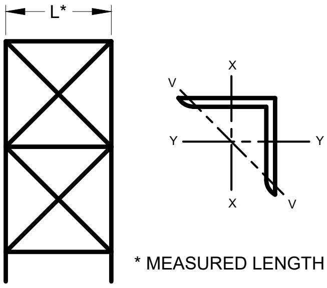
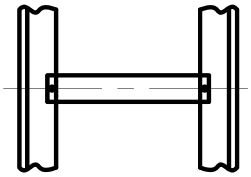
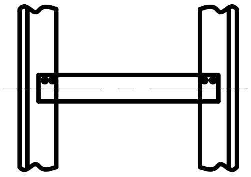
AE/r_{vv}
ED/r_{vv}
DC/r_{vv}
CB/r_{vv}

K-BRACINGS WITH AND WITHOUT SECONDARY MEMBERS

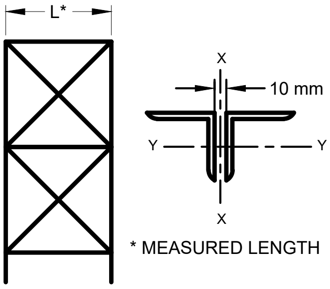
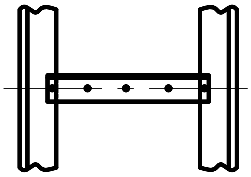
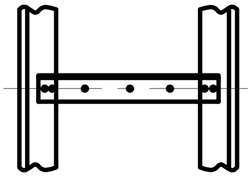
# The Corner stay should be designed to provide lateral support adequately

F-3 BELT MEMBERS WITH AND WITHOUT SECONDARY BRACINGS

EFFECTS OF END CONNECTIONS ON MEMBER CAPACITY

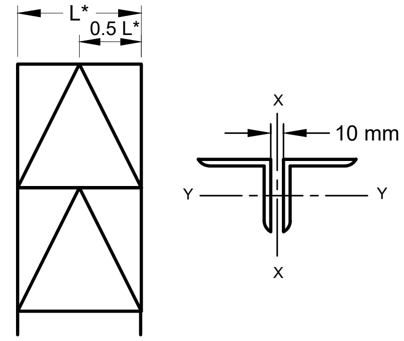
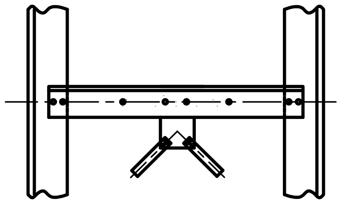
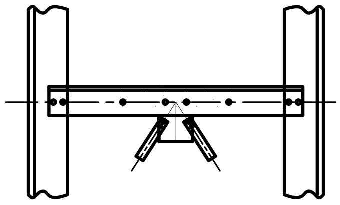
	METHOD OF LOADING/ RIGIDITY OF JOINTS	SLENDERNESS RATIO
 * MEASURED LENGTH	Tension system with compression strut : (Eccentricity in critical axis)	L/r_{vv} from 0 to 120 ($KL/r = 60+0.5L/r$)
	BRACING REQUIREMENT (SINGLE ANGLE MEMBERS) Single bolt connection, no restraint at ends	L/r_{vv} from 120 to 200 ($KL/r = L/r$)
	Multiple bolt connection,partial restraint at both ends	$L/_{vv}$ from 120 to 250 $KL/r = (46.2+0.615\ L/r)$

CONCENTRIC LOADING TWO ANGLE MEMBER

	METHOD OF LOADING/ RIGIDITY OF JOINTS	SLENDERNESS RATIO
 * MEASURED LENGTH	Tension system with compression concentric loading	L/r_{xx} or L/r_{yy} from 0 to 120 ($KL/r = L/r$)
	BRACING REQUIREMENT (TWO ANGLE MEMBERS) Single bolt connection, no restraint at ends	L/r_{xx} or L/r_{yy} from 120 to 200 ($KL/r = L/r$)
	Multiple bolt connection,partial restraint at both ends	L/r_{xx} or $L/_{yy}$ from 120 to 250 $KL/r = (46.2+0.615\ L/r)$

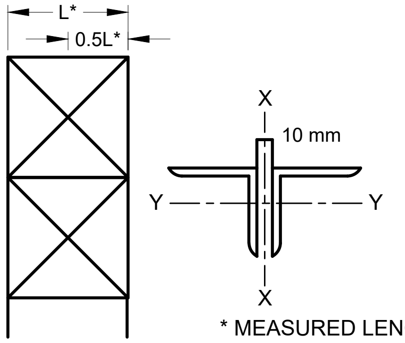
F-4 BELT MEMBERS IN K-BRACING SYSTEMS BACK TO BACK WITH AND WITHOUT SECONDARY BRACINGS

HORIZONTAL MEMBER OF K-BRACING-TWO ANGLE MEMBER

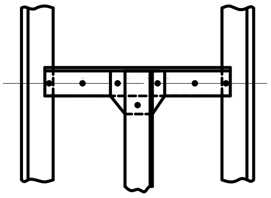
	METHOD OF LOADING/ RIGIDITY OF JOINTS	SLENDERNESS RATIO
 * MEASURED LENGTH	Tension Compression system with compression strut : Multiple bolt connection partial restraint at ends and intermediate	$0.5L/r_{yy}$ or L/r_{xx} from 120 to 250 $KL/r = (46.2 + 0.615L/r)$
	BRACING REQUIREMENT (TWO ANGLE MEMBER)	
	Concentric load at ends, eccentric loading at intermediate in both direction	$0.5L/r_{yy}$ or L/r_{xx} from 0 to 120 $KL/r = (30 + 0.75L/r)$
	Concentric load at ends, and intermediate	$0.5L/r_{yy}$ or L/r_{xx} from 0 to 120 $KL/r = L/r$

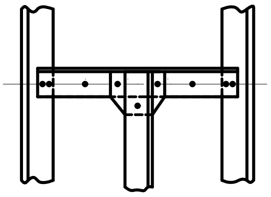
F-5 BELT MEMBERS BACK TO BACK WITH AND WITHOUT SECONDARY BRACINGS

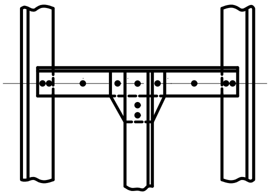
CONCENTRIC LOADING TWO ANGLE MEMBER, SUBDIVIDED PANELS OF A HORIZONTAL MEMBER



* MEASURED LENGTH



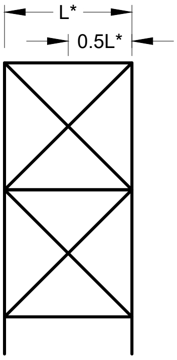


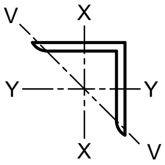


METHOD OF LOADING/ RIGIDITY OF PANEL	SLENDERNESS RATIO
Tension system with compression strut : Concentric loading	$0.5L/r_{yy}$ or L/r_{xx} from 0 to 120 (KL/r = L/r)
BRACING REQUIREMENT	
Single bolt connection, no restraint at ends and intermediate	$0.5L/r_{yy}$ or L/r_{xx} from 120 to 200 (KL/r = L/r)
Multiple bolt connection at ends.Single bolt connection at intermediate point	
Partial restraint at one end no restraint at intermediate	$0.5L/r_{yy}$ from 120 to 220 KL/r = (28.6+0.762 L/r)
Partial restraint at both ends	L/r_{xx} from 120 to 250 KL/r = (46.2+0.615 L/r)
Multiple bolt connection	
Partial restraint at ends and intermediate	$0.5L/r_{yy}$ or L/r_{xx} from 120 to 250 KL/r = (46.2+0.615 L/r)

F-6 BELT MEMBERS BACK TO BACK WITH AND WITHOUT SECONDARY BRACINGS

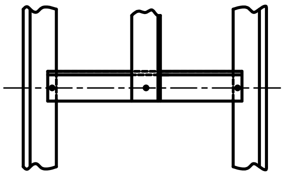
EFFECT OF SUBDIVIDED PANELS FOR THE HORIZONTAL MEMBER AND END CONNECTION ON MEMBER CAPACITY





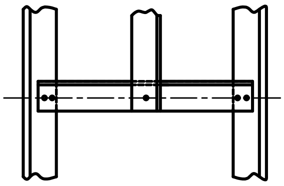
METHOD OF LOADING/ RIGIDITY OF JOINTS	SLENDERNESS RATIO
Tension system with compression strut : Eccentricity in critical axes	$0.5L/r_{vv}$ or L/r_{xx} from 0 to 120 ($KL/r = (60+0.5L/r)$)

* MEASURED LENGTH

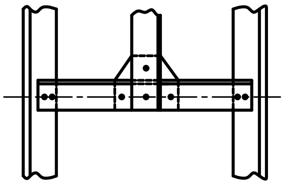


BRACING REQUIREMENT

Single bolt connection, no restraint at ends for intermediate	$0.5L/r_{vv}$ or L/r_{xx} from 120 to 200 ($KL/r = L/r$)
Multiple bolt connection at ends.Single bolt connection at intermediate point	



Partial restraint at one end, no restraint at intermediate	$0.5L/r_{vv}$ from 120 to 225 ($KL/r = 28.6+0.762\ L/r$)
Partial restraint at both ends	L/r_{xx} from 120 to 250 $KL/r = (46.2+0.615\ L/r)$



Multiple bolt connection	
Partial restraint at ends and intermediate	$0.5L/r_{vv}$ or L/r_{xx} from 120 to 250 $KL/r = (46.2+0.615\ L/r)$

F-7 EQUIVALENT SLENDERNESS RATIOS BASED ON END CONDITIONS

The equivalent slenderness ratios of bracing members, based on end conditions and types of members, are given at Table 6.

Table 6 Equivalent Slenderness Ratios of Bracing Members

(*Clause* F-7)

Sl No.	Type of Member and End Conditions	Value KL/r
(1)	(2)	(3)
Members with eccentric loading (for example, single angle or channels, round members with eccentric end plates, round members with flattened ends		
i)	Member with concentric loading at one end and normal eccentricities at the other end of the unsupported panel with values of L/r up to and including 120	$30 + 0.75\ L/r$
ii)	Members with normal framing eccentricities at both ends of the unsupported panel for value of L/r up and including 120	$60 + 0.50\ L/r$
Unrestrained condition (for example, single bolt connection)		
iii)	members unrestrained against rotation at both ends of the unsupported panel for value of L/r from 120 to 200	L/r
Partially restrained condition (for example, multiple bolt or welded connections to a stiffer member/component or group of members/components)		
iv)	Members partially restrained against rotation at one end of the unsupported panel for values of L/r over 120 up to and including 225	$28.6 + 0.762\ L/r$
v)	Members partially restrained against rotation at both ends of the unsupported panel for value of L/r over120 up to and including 250	$46.2 + 0.615\ L/r$

NOTES

1 Different equations may apply for each direction of buckling under consideration. The maximum effective slenderness ratio shall be used to determine the compressive strength.

2 The restrained member must be connected to the restraining member by at least two bolts.

3 The restraining member must have a stiffness factor I/L in the buckling plane (I = Moment of Inertia, L= Length) that equals or exceeds the sum of the stiffness factors in the buckling plane of the restrained members that are connected.

4 Angle members connected by one leg should have the holes located as close to the outstanding leg as feasible. Normal framing eccentricities at load transfer connection imply that connection holes are located between the heel of the angle and the center line of the framing leg.

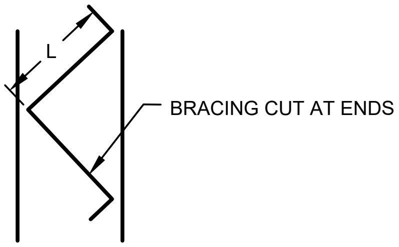
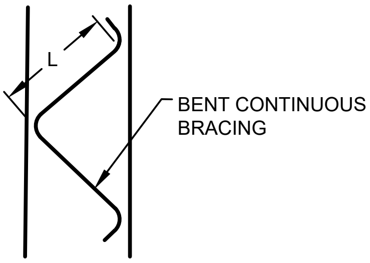
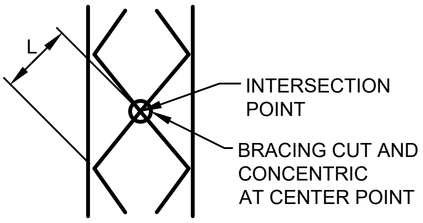
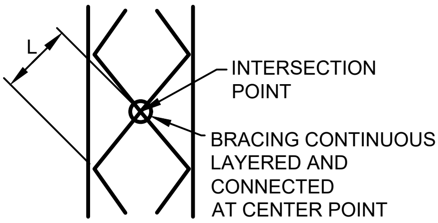
5 Where test and/or analysis demonstrates that any other type of bracing pattern is found technically suitable, the same may be adopted.

6 A single bolt connection shall not be considered as offering restraint against rotation. A multiple bolt connection properly detailed to minimize eccentricities shall be considered to offer partial restraint if connection is to a member having adequate flexural strength to resist rotation of the joint. Points of intermediate support shall not be considered as offering restraint to rotation unless they meet the criteria outlined under the slenderness ratio.

7 Limiting value of KL/r shall be as follows:

a) Main bracing members = 150
b) Members carrying computed stresses = 200
c) Redundant members and those carrying nominal stresses = 250
d) Members used for supporting ancillaries subjected to tension = 400

F-8 EFFECTIVE LENGTH FACTOR, *K*, FOR ROUND BRACING MEMBERS DIRECTLY WELDED TO LEGS

BRACING PATTERN	**SLENDERNESS RATIO OF BRACE MEMBER**		
	***L/r*<80**	**80 ≤ *L/r* ≤ 120**	***L/r*>120**
	K=1.0	$K=0.7+0.3(120-L/r)/40$	K=0.7
	K=1.1	$K=0.7+0.4(120-L/r)/40$	K=0.7
 L INTERSECTION POINT BRACING CUT AND CONCENTRIC AT CENTER POINT *See* Note 2	K=1.0	$K=0.75+0.25(120-L/r)/40$	K=0.75
 L INTERSECTION POINT BRACING CONTINUOUS LAYERED AND CONNECTED AT CENTER POINT *See* Note 2	K=1.1	$K=0.90+0.20(120-L/r)/40$	K=0.90

NOTES

1 *L* shall be determined using the panel spacing and the clear distance between the legs.

2 When tension force in the tension member of double braced pattern is less than 75 percent of the compression force in the compression member, the value of *KL* shall be determined based on a single braced condition.

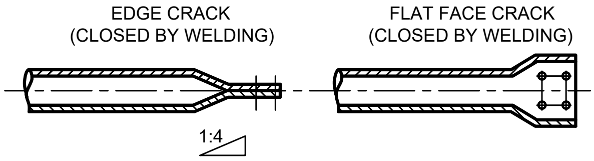
ANNEX G

[*Clause* 10.4.4.1 (a)]

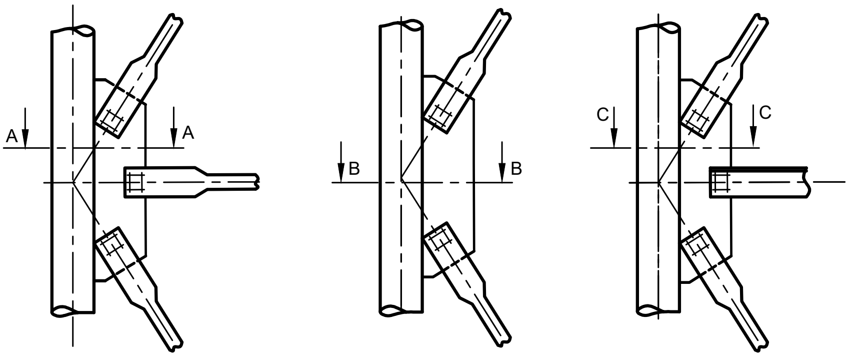
TUBULAR MEMBERS

G-1 CONNECTIONS IN TUBULAR MEMBERS

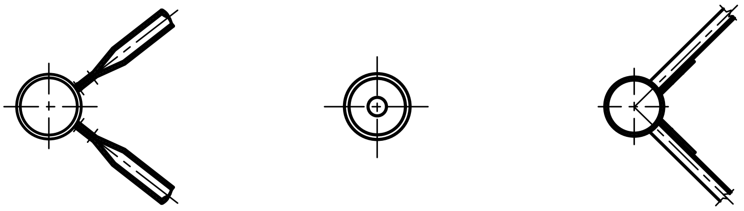
Bracing connection in tubulars members and its panel assembly are given in Fig. 16 and Fig. 17 respectively.



TYPICAL PIPE FLATTENING DETAIL



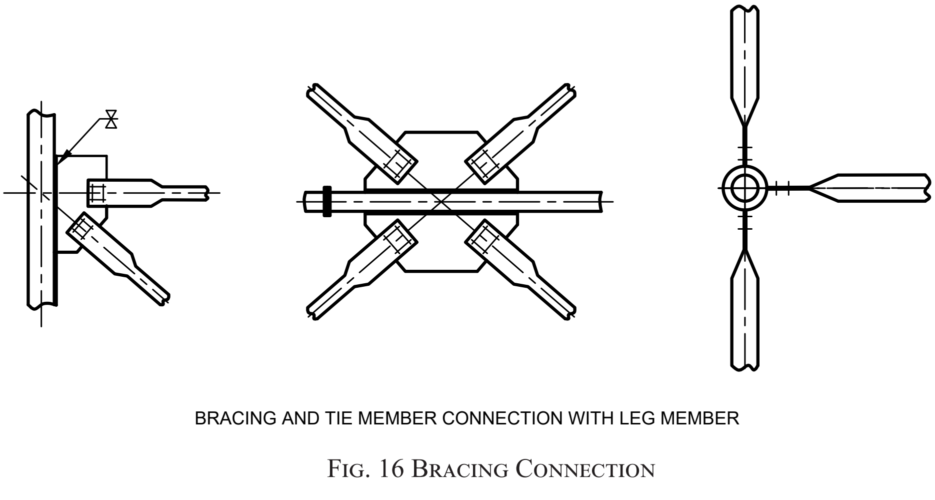
TYPICAL LEG BRACING CONNECTION



SECTION A-A

SECTION B-B

SECTION C-C



BRACING AND TIE MEMBER CONNECTION WITH LEG MEMBER

FIG. 16 BRACING CONNECTION

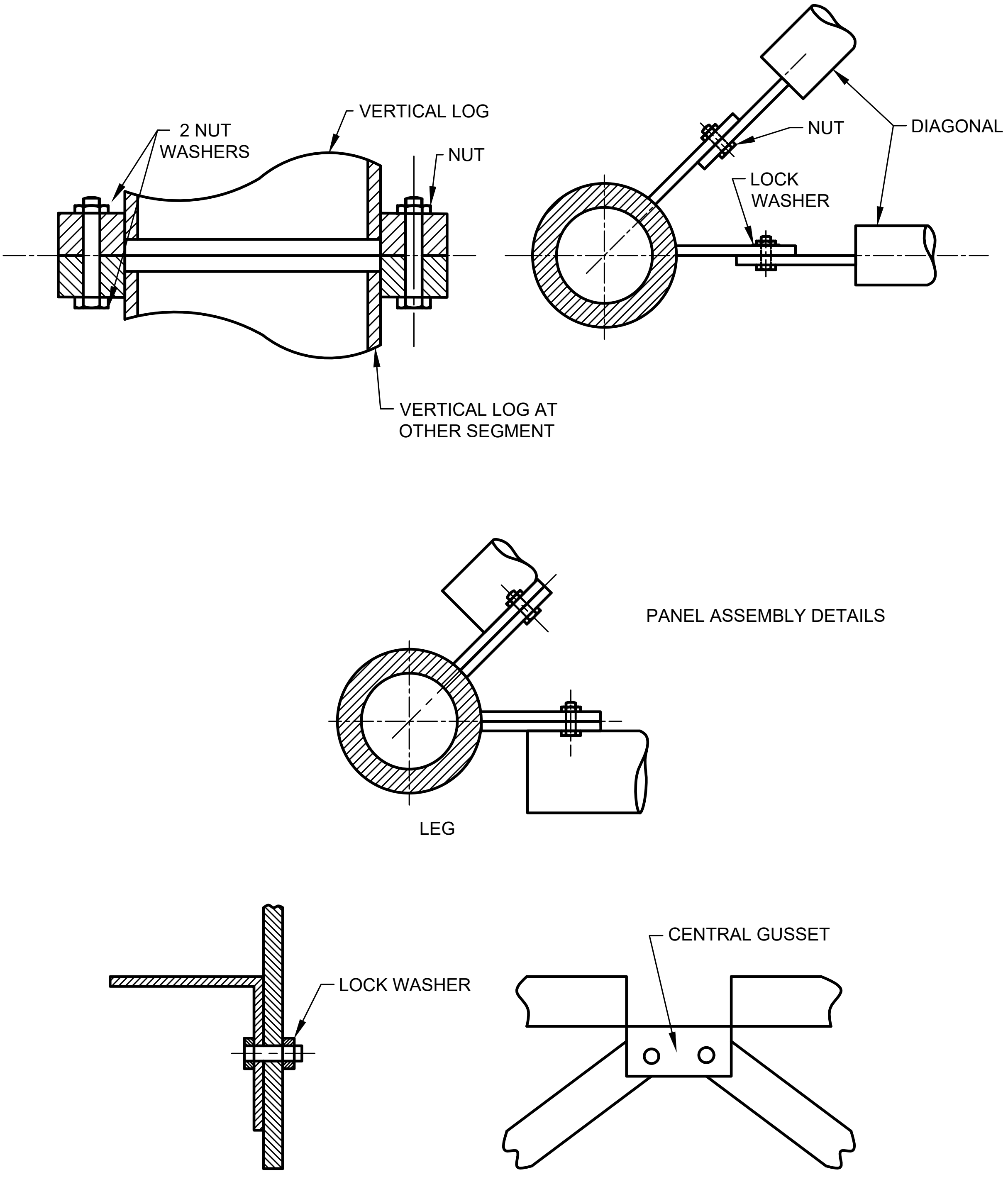
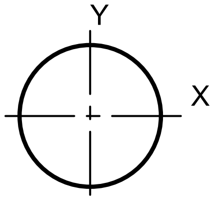
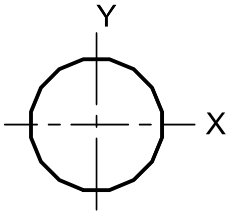
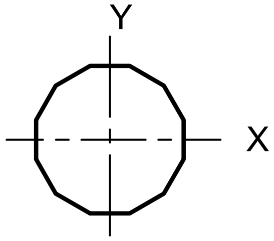
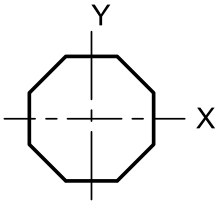
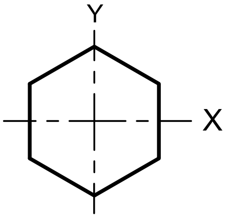


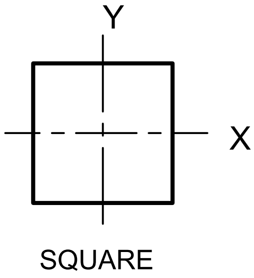
FIG. 17 PANEL ASSEMBLY

ANNEX H

(*Clause* 15.1)

PROPERTIES OF VARIOUS TUBULAR SECTIONS

 ROUND	$A_g = 3.14\,D\,t$ $I_x = I_y = 0.393\,D^3\,t$ $C_x = 0.5\,(D + t)\,Cos\,(\alpha)$ $C_y = 0.5\,(D + t)\,Sin\,(\alpha)$	$r = 0.354\,D$ $Max.\dfrac{Q}{I_t} = \dfrac{0.637}{Dt}$ $Max.\dfrac{C}{J} = \dfrac{0.637(D+t)}{D^3\,t}$
 HEXDECAGONAL	$A_g = 3.19\,D\,t$ $I_x = I_y = 0.403\,D^3\,t$ $C_x = 0.510\,(D + t)\,Cos\,(\alpha)$ $C_y = 0.510\,(D + t)\,Sin\,(\alpha)$ $\alpha = 11.25°$	$r = 0.356\,D$ $Max.\dfrac{Q}{I_t} = \dfrac{0.634}{D\,t}$ $Max.\dfrac{C}{J} = \dfrac{0.628(D+t)}{D^3\,t}$ $w = 0.199\,(D - t - 2\,BR)$
 DODECAGONAL	$A_g = 3.22\,D\,t$ $I_x = I_y = 0.411\,D^3\,t$ $C_x = 0.518\,(D + t)\,Cos\,(\alpha)$ $C_y = 0.518\,(D + t)\,Sin\,(\alpha)$ $\alpha = 15°$	$r = 0.358\,D$ $Max.\dfrac{Q}{I_t} = \dfrac{0.631}{D\,t}$ $Max.\dfrac{C}{J} = \dfrac{0.622(D+t)}{D^3\,t}$ $w = 0.268\,(D - t - 2\,BR)$
 OCTAGONAL	$A_g = 3.32\,D\,t$ $I_x = I_y = 0.438\,D^3\,t$ $C_x = 0.541\,(D + t)\,Cos\,(\alpha)$ $C_y = 0.541\,(D + t)\,Sin\,(\alpha)$ $\alpha = 22.5°$	$r = 0.364\,D$ $Max.\dfrac{Q}{I_t} = \dfrac{0.618}{D\,t}$ $Max.\dfrac{C}{J} = \dfrac{0.603(D+t)}{D^3\,t}$ $w = 0.414\,(D - t - 2\,BR)$
 HEXAGONAL	$A_g = 3.46\,D\,t$ $I_x = I_y = 0.481\,D^3\,t$ $C_x = 0.577\,(D + t)\,Cos\,(\alpha)$ $C_y = 0.577\,(D + t)\,Sin\,(\alpha)$ $\alpha = 30°$	$r = 0.373\,D$ $Max.\dfrac{Q}{I_t} = \dfrac{0.606}{D\,t}$ $Max.\dfrac{C}{J} = \dfrac{0.577(D+t)}{D^3\,t}$ $w = 0.577\,(D - t - 2\,BR)$

	$A_g = 4.00\, D\, t$ $I_x = I_y = 0.666\, D^3\, t$ $C_x = 0.707\ (D + t)\ Cos\ (\alpha)$ $C_y = 0.707\ (D + t)\ Sin\ (\alpha)$ $\alpha = 45°$	$r = 0.408\, D$ $Max.\frac{Q}{I_t} = \frac{0.563}{D t}$ $Max.\frac{C}{J} = \frac{0.500(D+t)}{D^3\, t}$ $w = (D - t - 2\, BR)$

NOTE – For Polygons, all properties (except w) assume a sharp cornered section.

where

a = Angle between the x-axis and the corner of the polygon (degrees);

D = Mean diameter (mm) = D_0 – t ;

D_0 = Outside diameter for round or across flats for polygonal members (mm);

t = Thickness (mm);

A_g = Gross area (mm^2);

I = Moment of inertia (mm^4);

C_x = Distance from x axis to point (mm);

C_y = Distance from y axis to point (mm);

r = Radius of gyration (mm);

Q/It (max) = Value for determining the maximum flexural shear stress (1/ mm^2);

C/J(max) = Value for determining the maximum torsional shear stress (1/mm^3);

J = Polar moment of inertia (mm^4);

w = Flat width of a side of a polygon (mm) (*see* Fig. 18);

BR = Effective bend radius (mm); if bend radius < 4t;

= Actual bend radius (mm); if bend radius > 4t, BR = 4t.

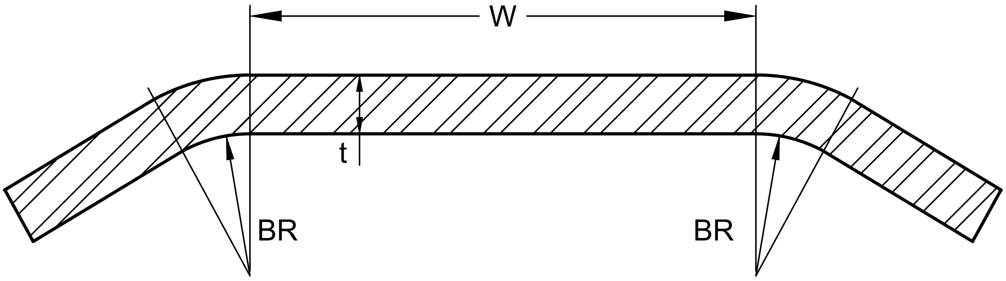


FIG. 18 FLAT WIDTH AND BEND RADIUS

ANNEX J

(*Clause* 17)

TYPICAL CHECKLIST FOR MAINTENANCE AND CONDITION ASSESSMENT OF TOWERS

(Means and methods for RF protection is not given)

(OK; NG – Needs maintenance or assessment)

ASSESSMENT	OK	NG
a) ***Super Structure and Appurtenances***		
1) ***Condition of the Structure*** i) Damaged leg, horizontal belts, main and secondary bracings ii) Missing members and loose members iii) Ladders, handrails, steps, platforms, guard rails iv) Loose, missing bolts, nut locking devices v) Undersized bolts, oversized bolt holes vi) Visible cracks in welded connections vii) Tower alignment – Tower plumb and twist		
2) ***Finish*** i) Condition of galvanizing/painting ii) Corrosion/rust in members, ladders and accessories iii) Aviation colour marking condition iv) Collection of water in members "(to be remedied eg, unplug drain holes etc)		
3) ***Lighting*** i) Condition of conduits, junction boxes, and fasteners (weather tight and secure) ii) obstructions in drain/vent openings iii) condition of electrical wiring iv) Bulb condition v) Controllers • Flasher • Photo control • Alarms		
4) ***Grounding*** i) Connections, corrosion ii) Lightning protection		
5) ***Antennas and Lines*** i) Antenna and antenna mount condition ii) Ice shield condition iii) Feed line condition iv) Hanger condition v) Secured to structure		
6) ***Other appurtenances (walkways, platforms, sensors, floodlights, etc.)*** i) Condition ii) Secured to structure		

ASSESSMENT	OK	NG
7) ***Guys*** i) Corrosion, broken strands, kinks etc. ii) Turnbuckles, thimbles iii) Service sleeves iv) Cable connecting end fittings and cable clamps v) Slippage or damage to the strands vi) Socket condition vii) Shackles and cotter pins		
8) ***Guy Tensions*** i) guy tensions shall be verified ii) Measure wind speed and temperature NOTE — Minor variations in guy tensions are expected. If significant, then the cause should be identified and rectified. Loosening during construction, exposure to extreme wind, disturbance to anchor, base settlement, or connection slippage may be some causes for variation in guy tensions. Tension variations at a single level are to be expected because of anchor elevation differences, construction deviations, and wind effects.		
b) ***Concrete Foundations***		
1) ***Ground Condition*** i) Settlement, movement or earth cracks ii) Erosion iii) Site condition (standing water, drainage, trees, etc.)		
2) ***Anchorage Condition*** i) Nuts and/or nut locking device (tightened) ii) Grout condition iii) Anchorages and/or anchor rod condition.		
3) ***Concrete Condition*** i) Cracking, spalling, or splitting ii) Chipped or broken concrete iii) Honeycombing iv) Low spots to collect moisture		
4) ***Guyed Mast Anchors*** i) Settlement, movement or earth cracks. ii) Backfill heaped over concrete for water shedding. iii) Anchor rod condition below earth (Maintain required structural capacity of anchor during exploration. Attachment to temporary anchorage may be required). iv) Corrosion control measures (galvanizing, coating, concrete encasement, cathodic protection systems, etc). v) Anchor heads clear of earth.		

ANNEX K

(*Clause* 18)

FIELD MAPPING OF STRUCTURES AND APPURTENANCES

K-1 MAPPING OF APPURTENANCES

The mapping of appurtenances shall provide sufficient dimensional data in order to calculate the effective projected area, weight and location of all appurtenances. The mapping of appurtenances shall include, as a minimum:

a) Inventory of existing antennas:
   1) Elevation;
   2) antenna type and dimensions/model number;
   3) support mount and location;
   4) spacing and orientation on cross-section; and
   5) corresponding transmission lines/cables.

b) Inventory of other appurtenances (such as climbing ladders, platforms, etc.):
   1) Elevation appurtenance type and dimensions, location; and
   2) spacing and orientation on cross-section.

K-2 MAPPING OF STRUCTURAL COMPONENTS

In order to perform an analysis of a structure, the structural configuration and the size of all structural members must be mapped in order to calculate wind loading and member capacities.

K-2.1 Self-Supporting Latticed Structures

a) Sketch of overall structure, numbering all sections.

b) The Configuration of each section:
   1) Section height;
   2) Panel height and number of panels;
   3) Configuration of the panels (X, X with horizontal, K);
   4) Face width (center to center of legs) at all taper change locations; and
   5) Sketch indicating the above for each typical section.

c) Member sizes for each section:
   1) Leg member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), solid round diameter, or angle size and thickness (60 degree or 90 degree).
   2) Diagonal member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), angle size, thickness and orientation (long leg back to back-LLBB, or short leg back to back-SLBB).
   3) Horizontal member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), angle size, thickness and orientation (LLBB, SLBB).
   4) Sub-brace member sizes (if applicable) — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), angle size, thickness and orientation (LLBB, SLBB).

K-2.2 Guyed Masts

a) Structure base type (fixed or pinned) and tapered or flat base.

b) Guy anchor dimensions: distance from base to guy anchors and their relative elevations to base and their orientation.

c) Sketch of overall structure numbering all sections. Locate and label all guy wire levels:
   1) The configuration of each section;
   2) Section height;
   3) Panel height and number of panels;
   4) Configuration of the panels (X, X with horizontals, K);
   5) Face width (centre to centre of legs) at all taper change locations; and
   6) Sketch indicating the above for each typical section.

d) Member sizes for each section:
   1) Leg member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), solid round diameter, angle size and thickness (60 degree or 90 degree).
   2) Diagonal member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), angle size, thickness and orientation (long leg back to back, LLBB, or short leg back to back, SLBB).
   3) Horizontal member sizes — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), solid round diameter, angle size, thickness and orientation (LLBB, SLBB).
   4) Sub-brace member sizes (if applicable) — That is, pipe diameter (outside diameter and wall thickness using ultra sound device), angle size, thickness and orientation (LLBB, SLBB).

e) Guy wire elevation, size and type for each guy level.

K-2.3 Connections

In order to perform a rigorous structural analysis of a structure the details of all structural connections must be mapped in order to calculate connection capacities. The mapping of the structure connections shall include, as a minimum, the following:

a) Member end connection details;
b) If bolted: number, type and size of end bolts and centre bolts;
c) Size and thickness of gusset plate with related details (hole sizes, edge distances, weld size and length);
d) If Welded: weld size and length of end and centre connections;
e) Splice connection details;
f) Number, type and size of bolts;
g) Size and thickness of splice plate with related details (hole sizes, edge distances, weld size and length,) and distance from panel intersection point;
h) Anchor rod type, size, number, and bolt circle;
j) Guy Assembly and connection details;
k) Preformed size/type, turnbuckle size, shackle size;
m) Socket size, pin size, link plate dimensions with related details;
n) Size and thickness of guy pull-off plate with related details (hole sizes, edge distances, weld size and length, stiffener size); and
p) Guy anchor head plate size, thickness, holes size, spacing, and edge distances of holes, shaft type, size and extension length and angle from horizontal plane and weld size and length of connection between shaft and fan plate.

K-2.4 Pole Structures

a) Sketch of overall structure numbering all sections;
b) Configuration of each section;
c) Section height – For flanged type, the length from splice to splice. For telescoping poles, the length from butt to butt;
d) If multi sided, number of sides;
e) Flat to flat dimension or diameter and circumference at top and bottom of each section;
f) Port hole opening size, reinforcing and location; and
g) Size of each section: wall thickness of each section.

ANNEX L

(*Clause* 7.5)

DESIGN WIND FORCE ON TYPICAL MICROWAVE ANTENNAS

L-1 Wind force data presented in Table 7 for typical microwave antennas (including grid antennas) are described in the antenna axis system having the origin at the vertex of the reflector. The axial force F_{AM}, acts along the axis of the antenna. The side force, F_{SM}, acts perpendicular to the antenna axis in the plane of the antenna and the wind vector. The twisting moment, M_M, acts in the plane containing F_{AM} and F_{SM} (*see* Fig.19).

In all cases, the magnitude F_{AM}, F_{SM} and M_M depend on the wind pressure of the wind, the projected frontal area of the antenna, and the aerodynamic characteristics of the antenna body. The aerodynamic characteristics vary with wind angle. The values of F_{AM}, F_{SM} and M_M shall be determined from the following equations.

$$F_{AM} = P_d\,C_A\,A$$

$$F_{SM} = P_d\,C_s\,A$$

$$M_M = P_d\,C_M\,A\,D$$

where

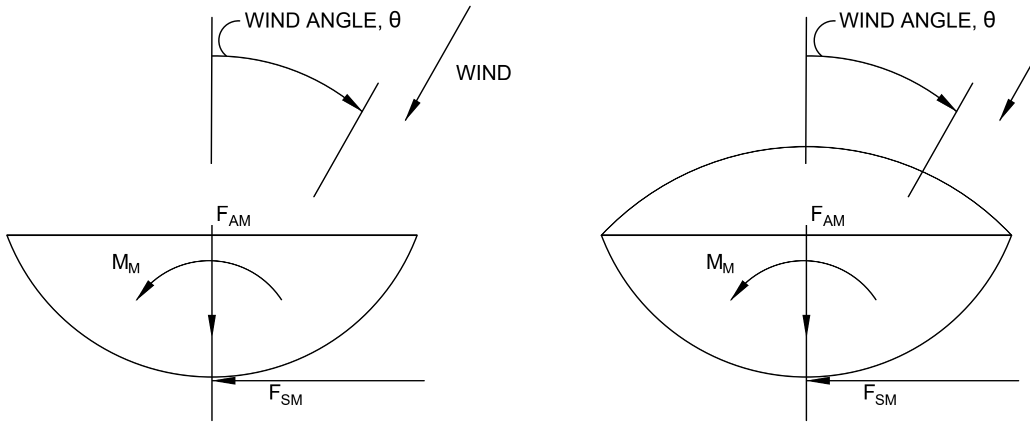
P_d = Design wind pressure (*refer* **7.5**)

C_A, C_S, and C_M are the coefficient contained in Table 7 as a function of wind angles, θ;

θ = wind angle, see Figure K for positive sign conventions;

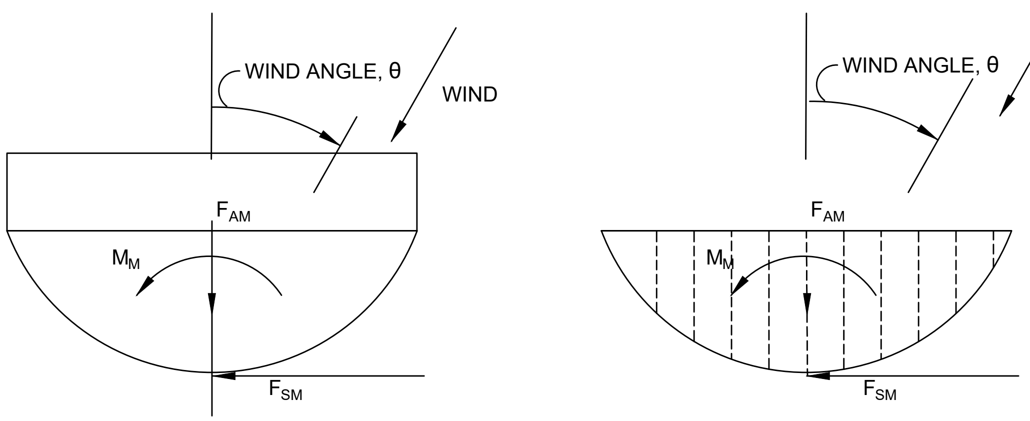
A = Outside aperture area of microwave antenna; and

D = Outside diameter of microwave antenna.



WITHOUT RADOME

RADOME



WITH CYLINDRICAL SHROUD

GRID

Fig. 19 Wind Forces on Typical Microwave Antennas

Table 7 Wind Force Coefficients for Typical Microwave Antenna

(*Clause* L-1)

WIND ANGLE θ (Degrees)	***Antenna without Radome***			***Antenna with Radome***			***Antenna with Cylindrical Shroud***			***Grid Antenna without Ice***		
	C_A	C_S	C_M	C_A	C_S	C_M	C_A	C_S	C_M	C_A	C_S	C_M
0	1.55	0.00	0.00	0.86	0.00	0.00	1.26	0.00	0.00	0.54	0.00	0.00
10	1.54	−0.05	−0.03	0.86	0.15	−0.08	1.26	0.10	−0.03	0.52	0.10	0.02
20	1.55	−0.05	−0.04	0.82	0.30	−0.11	1.25	0.18	−0.05	0.51	0.18	0.03
30	1.55	−0.03	−0.04	0.76	0.41	−0.11	1.21	0.23	−0.05	0.46	0.23	0.04
40	1.59	0.01	−0.05	0.66	0.49	−0.08	1.16	0.28	−0.05	0.41	0.26	0.04
50	1.66	0.09	−0.07	0.55	0.53	−0.04	1.09	0.30	−0.03	0.34	0.27	0.05
60	1.65	0.24	−0.09	0.42	0.50	0.00	0.95	0.37	−0.01	0.23	0.28	0.05
70	1.37	0.46	−0.01	0.31	0.46	0.05	0.67	0.48	0.02	0.13	0.27	0.06
80	0.76	0.38	0.10	0.23	0.44	0.10	0.27	0.58	0.07	0.04	0.25	0.05
90	−0.01	0.34	0.13	0.13	0.41	0.15	−0.11	0.63	0.10	−0.05	0.24	0.04
100	−0.40	0.38	0.13	0.03	0.39	0.17	−0.34	0.60	0.11	−0.12	0.27	0.05
110	−0.46	0.41	0.13	−0.07	0.37	0.16	−0.54	0.53	0.11	−0.19	0.29	0.05
120	−0.46	0.46	0.14	−0.16	0.35	0.16	−0.71	0.44	0.10	−0.27	0.28	0.05
130	−0.47	0.47	0.15	−0.29	0.32	0.14	−0.86	0.31	0.09	−0.34	0.26	0.05
140	−0.57	0.45	0.13	−0.41	0.30	0.09	−0.93	0.23	0.08	−0.41	0.23	0.04
150	−0.77	0.39	0.11	−0.52	0.27	0.05	−0.96	0.18	0.06	−0.48	0.20	0.04
160	−0.87	0.29	0.08	−0.60	0.23	0.02	−0.97	0.15	0.04	−0.55	0.16	0.03
170	−0.95	0.14	0.05	−0.66	0.15	0.01	−1.00	0.10	0.02	−0.59	0.09	0.01
180	−1.05	0.00	0.00	−0.69	0.00	0.00	−1.02	0.00	0.00	−0.59	0.00	0.00
190	−0.95	−0.14	−0.05	−0.66	−0.15	−0.01	−1.00	−0.10	−0.02	−0.59	−0.09	−0.01
200	−0.87	−0.29	−0.08	−0.60	−0.23	−0.02	−0.97	−0.15	−0.04	−0.55	−0.16	−0.03
210	−0.77	−0.39	0.11	−0.52	−0.27	−0.05	−0.96	−0.18	−0.06	−0.48	−0.20	−0.04
220	−0.57	−0.45	−0.13	−0.41	−0.30	−0.09	−0.93	−0.23	−0.08	−0.41	−0.23	−0.04
230	−0.47	−0.47	−0.15	−0.29	−0.32	−0.14	−0.86	−0.31	−0.09	−0.34	−0.26	−0.05
240	−0.46	−0.46	−0.14	−0.16	−0.35	−0.16	−0.71	−0.44	−0.10	−0.27	−0.28	−0.05
250	−0.46	−0.41	−0.13	−0.07	−0.37	−0.16	−0.54	−0.53	−0.11	−0.19	−0.29	−0.05
260	−0.40	−0.38	−0.13	0.03	−0.39	−0.17	−0.34	−0.60	−0.11	−0.12	−0.27	−0.05
270	−0.01	−0.34	−0.13	0.13	−0.41	−0.15	−0.11	−0.63	−0.10	−0.05	−0.24	−0.04
280	0.76	−0.38	−0.10	0.23	−0.44	−0.10	0.27	−0.58	−0.07	0.04	−0.25	−0.05
290	1.37	−0.46	0.01	0.31	−0.46	−0.05	0.67	−0.48	−0.02	0.13	−0.27	−0.06
300	1.65	−0.24	0.09	0.42	−0.50	0.00	0.95	−0.37	0.01	0.23	−0.28	−0.05
310	1.66	−0.09	0.07	0.55	−0.53	0.04	1.09	−0.30	0.03	0.34	−0.27	−0.05
320	1.59	−0.01	0.05	0.66	−0.49	0.08	1.16	−0.28	0.05	0.41	−0.26	−0.04
330	1.55	0.03	0.04	0.76	−0.41	0.11	1.21	−0.23	0.05	0.46	−0.23	−0.04
340	1.55	0.05	0.04	0.82	−0.30	0.11	1.25	−0.18	0.05	0.51	−0.18	−0.03
350	1.54	0.05	0.03	0.86	−0.15	0.08	1.26	−0.10	0.03	0.52	−0.10	−0.02

ANNEX M

(*Foreword*)

COMMITTEE COMPOSITION

Structural Engineering and Structural Sections Sectional Committee, CED 07

Organization	*Representative(s)*
In Personal Capacity (*Block 2, Flat 2A Rani Meyyammai Tower, MRC Nagar,RA Puram,Chennai 600028*)	Dr V. Kalyanaraman (***Chairman***)
Ashwathnarayana and Eswara, Chennai	Shri H. E. Sriprakash Shastry
Bhilai Institute of Technology, Durg	Dr Mohan Kumar Gupta
C.R. Narayana Rao, Architects and Engineers, Chennai	Dr C. N. Srinivasan Shri C. R. Arvind (*Alternate*)
Central Electricity Authority, New Delhi	Shri A. K. Jain Director (Transmission) (*Alternate*)
Central Public Works Department, New Delhi	Shri D. K. Garg Shri N. K. Bansal (*Alternate*)
Central Water Commission, New Delhi	Director - Gates (NW & S) Director - Gates Design (N & W) (*Alternate*)
Construma Consultancy Private Limited, Mumbai	Dr Harshavardhan Subbarao
Consulting Engineering Services (India) Private Limited, New Delhi	Shri S. Ghosh Dr B. C. Roy (*Alternate*)
CSIR-Structural Engineering Research Centre, Chennai	Dr G. S. Palani Dr Napa Prasada Rao (*Alternate*)
Engineers India Limited, New Delhi	Shri A. Sinha Shri Virender Goel (*Alternate*)
GAIL India Limited, New Delhi	Shri Shashi Ranjan Shri S. Ashish Vaidya (*Alternate*)
Gammon Engineers and Contractors Private Limited, Mumbai	Shri Rakesh Kumar Mehta Shri Abhilash Kumar Sudha (*Alternate*)
Garden Reach Shipbuilders and Engineers Limited, Kolkata	Shri Somnath Banerjee Shri Pranit Biswas (*Alternate*)
Geodesic Techniques Private Limited, Bengaluru	Shri Srinidhi Anantharaman Shrimati Jyoti C. Ubhayakar (*Alternate*)
Indian Institute of Engineering Science and Technology, Shibpur	Dr Saibal Kumar Ghosh Dr Subrata Chackraborty (*Alternate*)
Indian Institute of Technology Delhi, New Delhi	Dr Dipti Ranjan Sahoo Dr Alok Madan (*Alternate*)
Indian Institute of Technology Madras, Chennai	Dr S. Arul Jayachandran Dr Rupen Goswami (*Alternate*)
Indian Oil Corporation Limited, Noida	Shri T. Bandyopadhyay Shri P. V. Rajaram (*Alternate*)
Indian Road Congress, New Delhi	Secretary General Director (*Alternate*)
Institute for Steel Development and Growth, Kolkata	Shri Arijit Guha Shri Lakhamana Rao Pydi (*Alternate*)

Organization	*Representative(s)*
Jindal Steel and Power Limited, Gurugram	Shri Ashwani Kumar Shri Sanjay Nandanwar (*Alternate*)
Larsen and Toubro Limited, Chennai	Shri Suresh Babu Reddy
M N Dastur and Company Private Limited, Kolkata	Shri Shuvendu Chattopadhyay Shri Gargi Aditya Basu (*Alternate* I) Shri Mohua Chatterjee (*Alternate* II)
MECON Limited, Ranchi	Shri B. K. Pandey Shri J. K. Sarkar (*Alternate*)
Powergrid Corporation of India Limited, New Delhi	Shri Abhishek Shri Sumana Mukherjee (*Alternate*)
Military Engineer Services, Engineer-in- Chief's Branch, Integrated HQ of Ministry of Defence (Army), New Delhi	Brig Girish Joshi Lt Col Gaurav Kaushik (*Alternate*)
NTPC Limited, Noida	Shri Himanshu Kundu Shri Chander Shekhar (*Alternate*)
Research Designs and Standards Organization, Lucknow	Director (B & S)/SB-II Director (B & S)/SB-I (*Alternate*)
RITES Limited, Gurugram	Shri Anil Khanna Shri Pankaj Kumar (*Alternate*)
Steel Authority of India Limited, Ranchi	Shri Anurag Upadhyay Shri C. M. Chugh (*Alternate*)
Steel Re-Rolling Mills Association of India, Kolkata	Shri B. M. Beriwala Shri Rajesh Vijayavergia (*Alternate*)
STUP Consultants Private Limited, Kolkata	Shri A. Ghoshal Dr N. Bandopadhyay (*Alternate*)
Tata Consulting Engineers Limited, Mumbai	Ms Surovi Ganguly Shri Shriprasad T. (*Alternate*)
The Institution of Engineers (India), Kolkata	Shri S. H. Jain
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This Indian Standard has been developed from Doc No.: CED 07 (16865).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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Published by BIS, New Delhi